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EFFECTS OF REPEATED LOADING ON GRAVEL AND CRUSHED STONE BASE COURSE MATERIALS USED IN THE AASHO ROAD TEST

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Final Report

EFFECTS OF REPEATED LOADING ON GRAVEL AND CRUSHED STONE BASE COURSE MATERIALS USED IN THE AASHO ROAD TEST

by

John H. Haynes
Graduate Assistant

Joint Highway Research Project
Project: C-36-45H
File: 6-18-8

Purdue University
Lafayette, Indiana

June 21, 1961

Final Report

EFFECTS OF REPEATED LOADING ON GRAVEL AND CRUSHED STONE BASE COURSE MATERIALS USED IN THE AASHO ROAD TEST

TO: K. B. Woods, Director
Joint Highway Research Project

June 21, 1961

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 6-12-8
Project: C-36-45H

The final report titled "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test" is attached. It has been authored by John H. Haynes under the supervision of Professor E. J. Yoder of our staff. Mr. Haynes also utilized the report as his thesis for part of the requirements for the MSCE degree.

The report is submitted to the Board for the record and for approval for release of the information to the Highway Research Board and Test Road officials. The report should also be classified as confidential until approved for release by the Highway Research Board. Publication of the information is planned and will be developed in cooperation with Test Road officials.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Secretary

HLM:kmc

Attachment

cc: F. L. Ashbaucher
J. R. Cooper
W. L. Dolch
W. H. Goetz
F. F. Havey
F. S. Hill
G. A. Leonards
G. A. Hawkins (M. B. Scott)
J. F. McLaughlin
R. D. Miles
R. E. Mills
J. V. Smythe
C. E. Vogelgesang
J. L. Waling
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TABLE OF CONTENTS

Page

LIST OF TABLES iv

LIST OF FIGURES v

ABSTRACT vii

INTRODUCTION 1

REVIEW OF LITERATURE 6

PURPOSE AND SCOPE 10

PREPARATION OF MATERIAL 12

COMPACTION PROCEDURE 16

TESTING PROCEDURE 21

RESULTS 28

DISCUSSION OF RESULTS 52

- Compactive Energy and Relative Density 52
- Performance of Gravel 57
- Performance of Crushed Stone 62
- Comparison of Gravel and Crushed Stone 63
- Laboratory Comparison of AASHO Gradations 67
- Comparison of Laboratory and Field Results 70

SUMMARY 76

SUGGESTONS FOR FURTHER RESEARCH 79

BIBLIOGRAPHY 80

APPENDIX Deflection and Rebound Histories for Gravel and Crushed Stone Specimens 81

LIST OF TABLES

Table Page

1. Grain Size Distribution (AASHO Materials) 13

2. Grain Size Distribution of Laboratory Test Specimens 15

3. Summary of Relative Density Data 45

4. Change in Grain Size Distribution Due to Compaction 49

5. Comparison between Performance of Gravel and Crushed Stone Specimens of Equivalent Gradations 65

LIST OF FIGURES

Figure Page

1. Base Course Gradations Used in AASHO Road Test 14
2. Saturation Mold and Cell 18
3. Equipment Used for Repeated Load Tests 22
4. Typical Load-Time Trace for Two Load Cycles 24
5. Variation in Density with Compactive Effort-Gravel 29
6. Variation in Density with Compactive Effort-Crushed Stone 30
7. Deflection History of Gravel Specimens 32
8. Deflection History of Crushed Stone Specimens 33
9. Rebound History of Gravel Specimens 34
10. Rebound History of Crushed Stone Specimens 35
11. Variation of Deflection with Saturation Level-Gravel . . . 36
12. Variation of Deflection with Saturation Level-Crushed Stone 37
13. Variation of Rebound with Saturation Level-Gravel 38
14. Variation of Rebound with Saturation Level-Crushed Stone 39
15. Compactive Effort Required to Achieve Field Densities . . . 46
16. (a) Density Obtained at Various Compactive Efforts
(b) Compactive Effort Required to Achieve Field Densities . 47
17. Variation in Permeability with Density 51

A- 1. Deflection History of Gravel Specimens-6.2% Passing No. 200 Sieve 81

A- 2. Deflection History of Gravel Specimens-9.1% Passing No. 200 Sieve 82

LIST OF FIGURES (Continued)

A- 3. Deflection History of Gravel Specimens-11.5% Passing No. 200 Sieve 83

A- 4. Deflection History of Crushed Stone Specimens-6.1% Passing No. 200 Sieve 84

A- 5. Deflection History of Crushed Stone Specimens-9.1% Passing No. 200 Sieve 85

A- 6. Deflection History of Crushed Stone Specimens-11.5% Passing No. 200 Sieve 86

A- 7. Rebound History of Gravel Specimens-6.1% Passing No. 200 Sieve 87

A- 8. Rebound History of Gravel Specimens-9.1% Passing No. 200 Sieve 88

A- 9. Rebound History of Gravel Specimens-11.5% Passing No. 200 Sieve 89

A-10. Rebound History of Crushed Stone Specimens-6.2% Passing No. 200 Sieve 90

A-11. Rebound History of Crushed Stone Specimens-9.1% Passing No. 200 Sieve 91

A-12. Rebound History of Crushed Stone Specimens-11.5% Passing No. 200 Sieve 92

ABSTRACT

Haynes, John Hays. M.S.C.E., Purdue University, June 1961. "Effects of Repeated Loading on Gravel and Crushed Stone Base Course Materials Used in the AASHO Road Test." Major Professor: E. J. Yoder.

This thesis reports the results of a laboratory investigation of the behavior of gravel and crushed stone mixtures subjected to repeated loading. Cylindrical specimens were used and stressed triaxially to levels approximating those which would be found in the base course of a highway pavement. The repetitive loadings were applied with equipment developed in the laboratories of the School of Civil Engineering at Purdue University.

The materials used in the study were obtained from the site of the AASHO Road Test. The gravel and crushed stone with grain size distribution equal to that used in the Road Test and compacted to the mean density levels found in the field were the subject of primary interest. Variation in percent fines and degree of saturation were considered to have a great effect on these basic gradations. Therefore, the percent passing the number 200 mesh sieve was varied so that three mixtures of each material were obtained. These three mixtures were tested at three levels of saturation.

The effects of these variables on the deformation-rebound characteristics of the materials under repeated loads were studied. From these studies, the advantages and disadvantages of each material, concerning their value as base course materials, were found and are reported in detail. Also, a comparison between the laboratory and field performance of the two AASHO materials is presented.

INTRODUCTION

Some of the design concepts presently in use for determining the thickness of flexible pavements are based on the assumption that thickness is independent of the load transmitting medium. Other methods of design account for the stress-strain properties of the pavement components. Much research has been accomplished regarding the relative behavior of pavement materials but there is a continuing need for further investigations.

Several avenues of approach are available which enable the researcher to evaluate the effect of certain variables on pavement performance. The first deals with laboratory investigations into the properties of the materials themselves. Secondly, theoretical studies of stresses and strains in pavement structures have proven beneficial. A third approach is to evaluate, through performance surveys made on existing pavements, the factors which affect pavement behavior. Fourth, prototype test pavements yield valuable information under closely controlled test traffic.

In 1955, the Highway Research Board accepted the responsibility of building and testing a test road at Ottawa, Illinois. The AASHO Road Test, which is sponsored by the American Association of State Highway Officials, was built for the purpose of evaluating the basic behavior of flexible and rigid pavements under rigidly controlled conditions of test. The pavement was opened to traffic in the fall of 1958 and the scheduled traffic was terminated in the winter of 1960. This field testing program has produced much useful information about behavior of pavements under controlled truck traffic.

Four base course materials of various thicknesses were used in the flexible pavements at special locations. They included asphalt-stabilized, portland cement-stabilized, crushed stone and gravel material. The research reported herein was initiated for the purpose of developing a method of test which could be used by design engineers to evaluate the relative strength properties of the four base course materials. However, as is well known, these materials have widely varying physical characteristics which are difficult to evaluate with a single test. Cement-stabilized soil-aggregate is strong in compression but may be relatively weak when subjected to bending under certain conditions of load. Gravel and crushed stone cannot be stressed in tension and shear stresses are important characteristics in their evaluation. Bituminous-stabilized soil-aggregate occupies an intermediate position, relative to its properties of strength. While it may have resistance to compression, it will also resist bending stresses. Due to its flexibility, bituminous-stabilized soil-aggregate can resist permanent cracking caused by flexural stresses. This latter material possesses another very important property; variation in strength resulting from temperature changes. As the temperature of the asphalt contained in the matrix of the soil-aggregate is raised, the resistance to deformation of the bituminous-stabilized base will decrease. Increased temperature has little known effect on the strength of the other three materials.

Freezing-and-thawing affects the four materials to varying degrees. A freeze-thaw test is the commonly accepted method used for determining the quality of cement-stabilized materials. Engineers assign great importance to the ability of this material to withstand deterioration caused by the freezing and thawing of the entrapped moisture. Capillary water also contributes to the relative stability of these mixtures. The effect of capillary

water is also related to the quantity of fine material in the mixture, the relative density of the compacted base, permeability and frost susceptibility of the mixture and the climate of the area in which the pavement is constructed. In the case of soil-aggregate mixtures, capillary water and permeability are dominating factors since the stability of these mixtures is directly related to their degree of saturation. In contrast, for bituminous and cement-stabilized mixtures, permeability is of less consequence since water which enters the voids does not appreciably decrease the strength of these mixtures.

From the above brief discussion of some of the more important factors involved in an evaluation of the relative strengths of these materials, it is seen that many complex variables are present and that it is difficult to assign a comparative index of strength to these materials.

The AASHO Road Test project was so constructed that all of these base materials, used in special study sections and constructed to varying thicknesses, could be studied in detail under varying conditions of load and wheel configurations. The bases in these special sections were built in a wedge shape, the thickness of base thus varying from about two inches to 18 inches at the ends of the sections.

The pavement was opened to traffic during the fall of 1958. During February and March of 1959, some map cracking became pronounced in the pavements overlying the gravel bases. The magnitude of this cracking apparently was not influenced by the thickness of base. During the same period, the pavements overlying the other three base materials also exhibited some distress, the amount being influenced by base thickness and wheel load. During the month of March, 1959, many of the gravel sections, regardless of thickness, failed and were taken out of the testing program. An

important feature of the distress patterns is that the failure of pavements built with the gravel base was not associated with base thickness to the extent that distress of the other pavements was associated with a variation in base thickness. This leads to the speculation that some inherent factor relating to the gravel itself is prime suspect for causing pavement breakup.

Due to the complexity of the overall problem of evaluating each of the four materials in the laboratory, it was decided that the best approach would be to limit the study to the two base materials which most closely follow equal behavior patterns under varying weather and stress conditions. These materials are the gravel and crushed stone base gradations.

The physical process of progressive failure of the pavement sections underlain with gravel and crushed stone base courses was discussed with personnel of the AASHO Road Test. From these discussions, it was postulated that the distress of the flexible surfaces built over the gravel and crushed stone bases was due to two or three principle causes. The first was the inherent strength of the base materials which permitted consolidation of the base in the wheel paths and localized shear failures. The second, and indeed the most subtile, was the rebound properties of the compacted base materials which caused the asphaltic surface to crack due to fatigue. Many sections failed primarily through a process of development of hair cracks in the pavement, deteriorating to map cracking, and finally complete disintegration as large chunks of the pavement's surface were picked up by the vehicles' wheels and removed from the surface. Climatic conditions, in particular frost action, were recognized as contributing factors to the progressive distress of the pavements. Due to the interaction of these processes, it was necessary to find some method of test which measured all the suspected properties of the material.

It was considered desirable to evaluate these properties under controlled laboratory conditions in which the deformation and rebound properties could be recorded under an accelerated repeated stress history. Testing the materials under controlled triaxial stress conditions seemed to be the most appropriate method of obtaining an indication of the magnitude of these properties. As the thickness of the base courses varied from two to 18 inches, the results of a triaxial study were not considered to be applicable to some of the thinner sections but were considered acceptable as an indication of the stability of the thicker sections.

REVIEW OF LITERATURE

A study of previous research concerning the field of repeated loads in granular materials revealed the fact that only limited studies have been made. A greater amount of information, however, is available on dynamic loading of cohesive soils.

Tschebotarioff and McAlpin (1) in 1947 reported studies using repeated loads on the surface of compacted sands, clays and sand-clay mixtures. These tests were performed using a modified CBR test procedure. They found that a uniformly graded sand was particularly susceptible to excessive deformation when subjected to this type of loading as compared to the deformation of identical specimens loaded with an equal static force. They reported that the magnitude of deformation of the compacted clay tested under repeated loads was very similar to the magnitude produced using static load conditions. Sand-clay mixtures occupied an intermediate position between these two extremes. The difference in deformation caused by repeated loads as compared to static loads appeared to decrease in magnitude as larger proportions of clay were added to the sand. They found that even small admixtures of clay appreciably reduced the deformation produced by repeated loading, especially at low water contents.

In 1955, Seed, Chan and Monismith (2) reported the results of repeated load tests on silty clay soils using a lever-type loading frame. This was the first report from an extensive research program carried on at the University of California concerning the deformation characteristics of compacted cohesive materials under repeated load conditions. They found that

the deformation of silty clay specimens subjected to repeated stress applications in a triaxial cell was considerably greater than that of similar specimens subjected to a static load of the same magnitude.

Seed and McNeill (3) compared the deformation characteristics of a silty clay and a clayey silt which had similar stress-strain relationships in static triaxial compression tests. In repeated load tests, it was shown that, even though the two specimens deformed almost equally under the first load application, after 1,000 applications the clayey silt had deformed about fifty percent more than the silty clay. Reference is made in the paper to the effects a highly resilient soil might have on the fatigue of asphalt pavements. It was shown that the clayey silt exhibited much higher resiliency under equivalent repeated stress conditions than did the silty clay. These materials were also tested under varying degrees of compaction. It was noted that large changes in deformation occurred in the repeated load tests for small changes in degree of compaction. Specimens compacted to 95 percent of modified AASHO deformed only three percent of their original height after 1,000 stress applications whereas specimens compacted to 91 percent of modified AASHO deformed more than three times this amount at 1,000 load cycles. Further the resilient strain increased from 0.24 percent to 0.29 percent as the degree of compaction was varied from 95 percent to 91 percent.

Havers and Yoder (4) reported in 1957 the design and operation of a new apparatus for testing materials under repeated loading conditions. This equipment eliminated the effects of impact and mechanical linkages which had impaired the usefulness of previously designed equipment. An electrical timer was used to send signals to a solenoid valve which passed compressed air into the upper chamber of a compression cylinder. This air

applied pressure to the cylinder's piston which loaded the material placed below the cylinder. In 1959, Seed and Fead (5) reported the design of a similar device. An improvement was made in their equipment by placing the pressure cylinder below the material being tested so that the weight of the piston could be counterbalanced, thus eliminating residual pressures caused by the weight of the piston.

Yoder and Woods (6) investigated the compaction and strength characteristics of soil-aggregate mixtures using the standard CBR test as a measure of each mixture's strength. A selected gradation of aggregate was used; density and strength comparisons were made by varying the percentages of soil in each mixture. It was found that for a fixed gradation of aggregate, there was an optimum soil content at which maximum density was obtained. However, it was found that the soil content for maximum strength was somewhat less than that found for maximum density. Miller and Sowers (7) also investigated the effects of varying the proportions of aggregate and soil on the strength of soil-aggregate mixtures. The values of cohesion and internal friction for each mixture were determined from the results of static triaxial tests. It was shown that as the amount of soil in a mix was increased, the specimen's angle of internal friction dropped noticeably indicating that some of the soil was trapped between the aggregate particles which in turn prevented complete grain to grain contact of the aggregate. As more soil was added, the angle of internal friction became essentially constant. It was also found that cohesion increased as the soil content was increased. These tests showed that maximum bearing capacity (using a bearing capacity formula derived by the author and using the measured values of cohesion and internal friction) resulted with the soil-aggregate mixture that also produced the highest density in the compaction tests.

Seed and Monismith (8) studied the relationship between density, degree of saturation and stability of a sandy clay and a silty clay soil. The Hveem Stabilometer was used to evaluate the stability of these materials. It was found that for saturated conditions, as the density of the soil was increased its stability also increased. However, for partially saturated soils, the desirable density for maximum stability depended on the water content of the soil and it was found that increased density in some cases may have a deleterious effect on stability.

PURPOSE AND SCOPE

The research described in this thesis was initiated for the purpose of investigating in the laboratory the performance characteristics, under repeated stress applications, of the gravel and crushed stone materials used as base courses in the special flexible pavement wedge sections of the AASHO Road Test.

Two variables, percent fines and degree of saturation, were used in this study. Three percentages of fines were used, 6.2, 9.1 and 11.5 percent passing the number 200 mesh sieve. The latter two values were the mean percentages which were actually used in the AASHO test sections; 9.1 percent fines in the gravel and 11.5 percent fines in the crushed stone gradation. Three levels of saturation were used; 70, 85 and 100 percent. A constant dry density corresponding to those used in the test sections was maintained. The dry densities used for the gravel and crushed stone were 145 pcf and 141 pcf, respectively.

The specimens were tested under repeated triaxial stress conditions. A conventional type triaxial cell was used. The test specimens were eight inches high and four inches in diameter. The lateral confining pressure was kept at a constant value of 15 psi since this corresponds approximately to the level of lateral pressure which exists under the test wheel loads. A repeated deviator stress of 55 psi was used thus making the total axial stress under full load 70 psi. It was hypothesized that pore pressures developed under rapid field loading and unloading conditions cannot be

dissipated prior to the release of each stress cycle and therefore it was decided to perform all tests under undrained conditions. The above stress levels and undrained conditions were maintained throughout the entire test program.

Each specimen was subjected to 100,000 load cycles or to the number of cycles required to deform the specimen axially at least one-half inch whichever occurred first. Readings were taken during each test of the total axial deformation under full load and the magnitude of rebound from full load to residual load. These measurements were taken at periodic intervals throughout each test.

PREPARATION OF MATERIAL

The base course materials used in this study were obtained from the site of the AASHO Road Test, the gravel was obtained directly from the roadbed and the crushed stone from a stock pile. The gravel was a glaciated material whereas the stone was crushed dolomitic limestone. The grain size distribution of the two materials used in the AASHO Road Test is shown in Table 1 and Figure 1. It will be noted that the crushed stone mixture contained particles up to one and one half inches in diameter and the gravel mixture contained material as large as one inch in diameter. In accordance with current practice (9), the maximum size aggregate that should be used in the triaxial test, using four inch diameter specimens, is three quarters of an inch. Therefore, all aggregate larger than three quarters of an inch in diameter was removed from the material and replaced by an equivalent weight of 3/4 to 1/2 inch material. The revised AASHO gradations for the two materials are shown in Table 1.

As it was desired to investigate the behavior of each material with varying quantities of material finer than the number 200 mesh sieve, the gravel and crushed stone were separated into fractions and recombined as desired. The bulk material was first dry sieved into eight fractions and then the coarse fractions were wet sieved to remove all surface fines. The desired gradations were then manufactured, by increasing or decreasing the percent fines, until the three desired gradations of 6.2, 9.1 and 11.5 percent passing the number 200 sieve were obtained. The resulting gradations for the gravel and crushed stone are shown in Table 2.

TABLE 1

GRAIN SIZE DISTRIBUTION

Sieve Size	Gravel		Crushed Stone	
	Actual AASHO Gradation (% Passing)	Revised AASHO Gradation (% Passing)	Actual AASHO Gradation (% Passing)	Revised AASHO Gradation (% Passing)
1 1/2"	100.0	-	100.0	-
1"	99.0	-	90.0	-
3/4"	-	100.0	80.0	100.0
1/2"	74.0	74.0	68.0	68.0
#4	49.0	49.0	50.0	50.0
#8	-	40.8	-	38.5
#10	-	-	36.0	-
#16	35.0	35.0	-	30.5
#40	23.0	-	21.0	-
#50	-	19.2	-	18.2
#100	12.8	12.8	14.5	14.5
#200	9.1	9.1	11.5	11.5

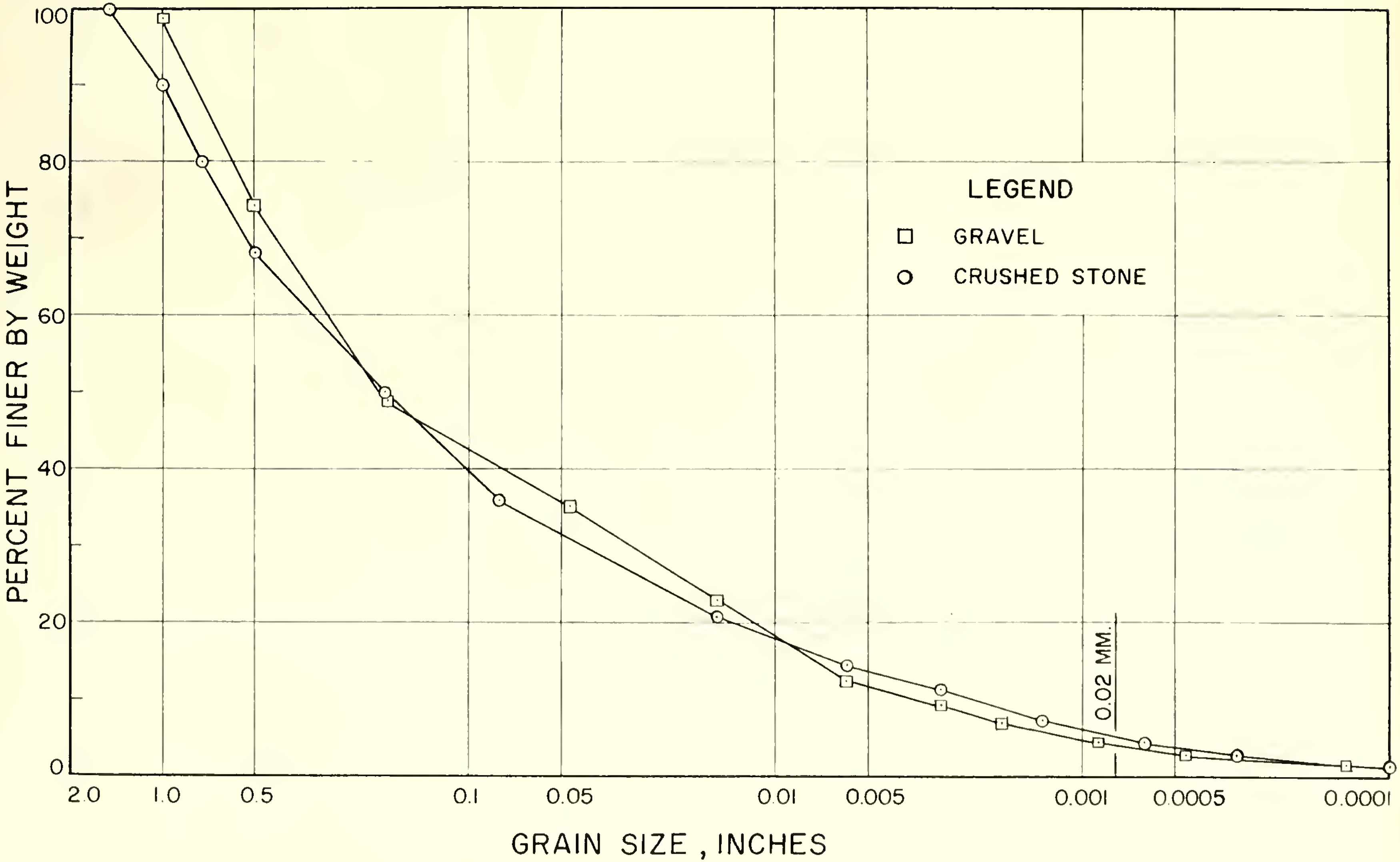

FIG. I BASE COURSE GRADATIONS USED IN AASHO ROAD TEST

TABLE 2

GRAIN SIZE DISTRIBUTION OF LABORATORY TEST SPECIMENS
(Percent Passing)

Sieve Size	Gravel Gradations (% Passing)			Crushed Stone Gradations (% Passing)		
	6.2	9.1	11.5	6.2	9.1	11.5
3/4"	100.0	100.0	100.0	100.0	100.0	100.0
1/2"	73.2	74.0	74.7	66.1	67.1	68.0
#4	47.4	49.0	50.4	47.0	48.6	50.0
#8	38.9	40.8	42.4	34.8	36.8	38.5
#16	32.9	35.0	36.7	25.3	27.6	29.5
#50	16.6	19.2	21.3	13.3	16.0	18.2
#100	10.0	12.8	15.1	9.4	12.2	14.5
#200	6.2	9.1	11.5	6.2	9.1	11.5

COMPACTION PROCEDURE

The field dry densities used in the AASHO Road Test were 145 pcf and 141 pcf, for the gravel and crushed stone respectively. These density levels were used throughout the laboratory testing program. Therefore, the compactive effort required to attain the desired density for each gradation of the two materials had to be determined. As a degree of saturation of 70 percent was to be the lowest moisture level used in the tests, the moisture content which corresponded to this degree of saturation for the desired density was used as the molding moisture content in all the compaction tests. The moisture content was calculated using measured values of specific gravity and the value of the required density. The specific gravities of the gravel and crushed stone gradations were 2.74 and 2.81, respectively. It was postulated that moisture curing time would have a large influence on the compacted dry density; therefore, a compaction procedure was developed which allowed four days between the time that water was added to the dry mixture and the mixture was compacted.

The standard AASHO 5.5 pound compaction hammer was used in the compaction of all specimens. A split compaction mold four inches inside diameter and eight inches high with removable base plate was used for molding the specimens. During compaction, it was fitted with a collar extension which retained the material for the final layer. Each specimen was compacted in four layers of equal thickness. By compacting four or five samples at increasing compactive effort, a semilog plot could be drawn which related

compactive effort and dry density. A wide range of compactive efforts was used, so that a fairly accurate straight line resulted and the required compactive effort for the desired density was determined from this line. These curves for the various gravel and crushed stone mixtures are shown in Figures 5 and 6 (pages 29 and 30). The compactive effort required to produce the desired dry density for each mixture was then used to compact all the subsequent specimens. Due to the coarseness of the materials, the compacted specimens were difficult to level. The large particles which protruded above the top of the mold were removed and were replaced with smaller ones in order to obtain a smooth surface. After leveling, the specimen and mold were weighed and the dry unit weights calculated.

The compacted specimen was next placed in the aluminum saturation mold shown on the right in Figure 2. The mold consisted of a four inch inside diameter split cylinder having a top and bottom which, when assembled, had a height of eight inches. Holes one-eighth inch in diameter were drilled through the cylinder walls on one inch centers so that water could easily flow through the openings.

First, the bottom plate of the saturation mold was placed on the leveled surface of the specimen and the compaction mold was inverted. The base plate of the compaction mold was then removed and the split mold was loosened so it would slide down over the pedestal attached to the bottom plate of the saturation mold. The split mold was then removed from the specimen. If the specimen was to be saturated, a sheet of wet filter paper was wrapped around the sides of the specimen. This prevented the fines near the surface from being sucked out of the sample when vacuum was applied. If the specimen was to be tested at its compacted degree of saturation, this step was omitted.

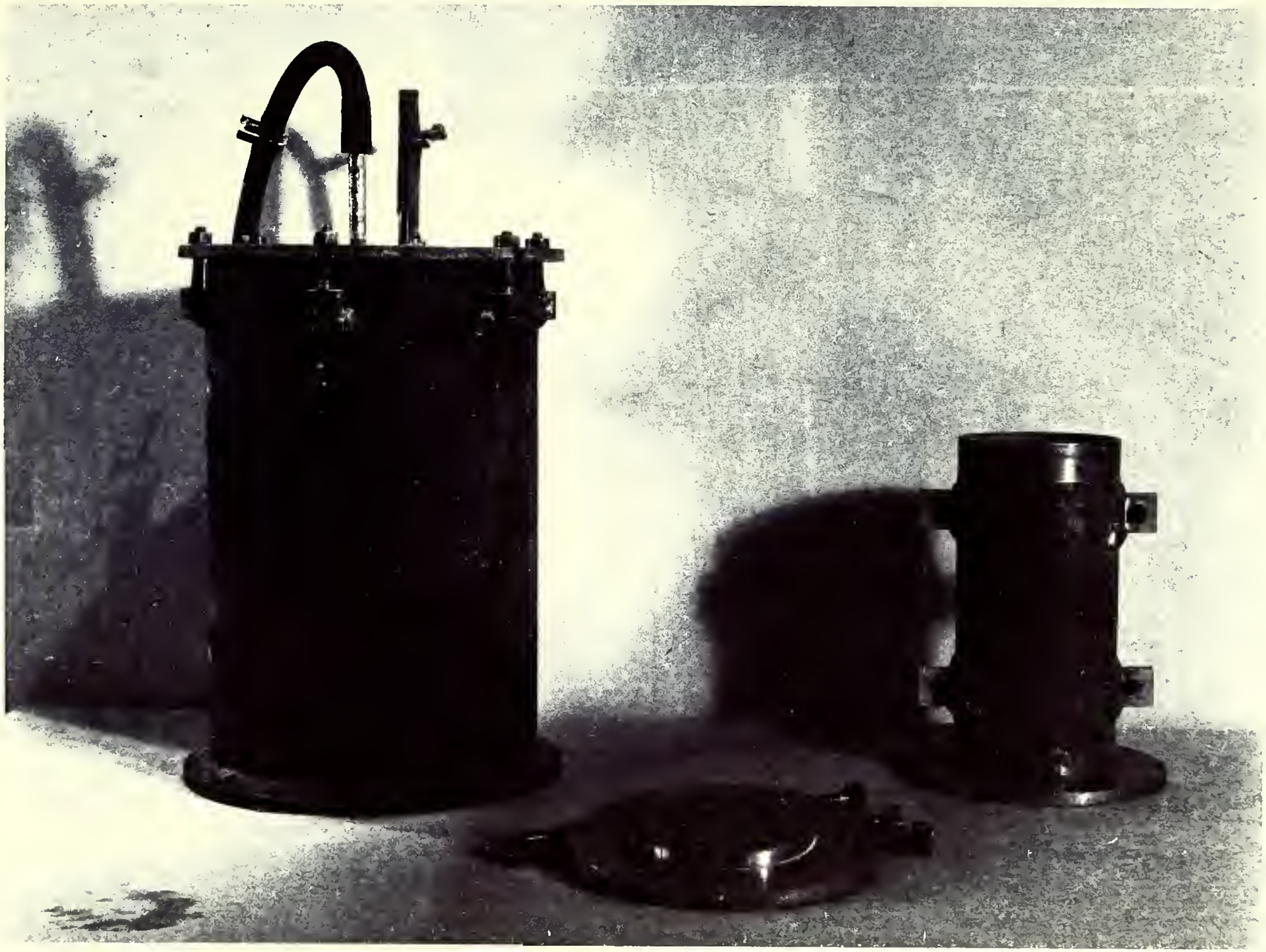

FIG. 2 SATURATION MOLD AND CELL

The split saturation mold was next placed around the specimen and tightened. The top was placed on the mold and the three bolts were secured. The saturation mold, containing the wet specimen, was then weighed so the wet weight of the specimen would be known. If the specimen was to be tested at its molding moisture content, it was wrapped in a plastic sheet to prevent loss of moisture and placed in a moisture room for a period of two days. If the moisture level was to be raised, the saturation mold was next lowered into the saturation cell shown on the left in Figure 2.

The saturation cell consisted of an eight inch inside diameter metal cylinder closed at the bottom. The cell had a metal top plate which was sealed with a flat rubber gasket and six bolts. After the top plate was secured to the cell, the vacuum pump was started and the air in the cell was evacuated. The vacuum pump was left running for 30 minutes, then the outlet line was closed and water was allowed to enter the cell from an adjacent water reservoir through the inlet line. The inlet line was closed after the water in the cell completely covered the saturation mold. The cell was left in this state of partial vacuum for an additional 30 minutes, then the vacuum pump was again started and the outlet line opened. After a period of 30 minutes, the line was again closed and the cell was left under vacuum for about 24 hours.

For those specimens to be tested at a degree of saturation approximating 100 percent, the vacuum was now released after 24 hours. The saturation mold was left in the cell until ready for use so that the surface of the specimen would not lose any moisture through evaporation. The mold was then removed from the cell and weighed. For those specimens to be tested at 85 percent saturation, the total saturation time was reduced to about

four hours. Since the dry weight of the specimen contained in the saturation mold was known, the weight of the specimen, mold and water required to produce 85 percent saturation was readily calculated. After the correct degree of saturation was attained, the specimen and mold were wrapped in plastic and stored for an additional day in the moisture room. During this period, the specimens were rotated at least twice so that the water would remain equally distributed throughout the specimen.

When the specimen was required for testing, the top plate of the saturation mold was removed and a solid round aluminum plate one quarter inch in thickness was then placed on the leveled surface of the specimen. This plate was designed so that it would fill the space left by the pedestal when the top plate was removed and would be flush with the sides of the saturation mold. The mold was then inverted so that this plate now rested on the flat surface of the table. The bottom plate of the mold was removed and the split mold was loosened. The two halves of the split mold were then removed and the filter paper, if used, was also removed. The specimen was now ready to be transferred to the base plate of the triaxial cell, the thin round aluminum plate serving as a support during this operation.

The moisture content used in the field during compaction of the gravel base material was higher than that used in compacting the laboratory specimens. Due to this variation, the clay binder particle orientation after laboratory compaction undoubtedly differed from that which existed in the field. It is probable that this had some effect on the deformation and rebound properties of the materials. It should also be recognized that the discussion of variation in degree of saturation with height of specimen (to be found in the Results section) is based on uniform density throughout the specimen. Due to compaction limitations, the density undoubtedly varied somewhat throughout the specimens.

TESTING PROCEDURE

The apparatus used for the repeated load tests is shown in Figure 3. This machine was designed and built by Havers (4) in the laboratories of the School of Civil Engineering at Purdue University. A short discussion of the basic operation of the machine is given below so that the subsequent discussions will be fully understood.

Compressed air at line pressure flowed through an air pressure regulator which reduced the pressure to about 14.5 psi. A surge tank was provided in the line so that a large volume of air could be stored under pressure near the point of use. This reduced the buildup time of the pressure on the specimen from no load to full load. The air then passed through an air filter which removed moisture and foreign particles. From this air filter, the air flowed through an air line lubricator and then entered an electrically controlled solenoid valve. This valve was connected to an eight inch pressure cylinder. As can be seen in Figure 3, the air cylinder is mounted vertically to the loading frame and the piston is in direct contact with a proving ring. The base of the proving ring rests on the piston of the triaxial cell which imparts the load to the specimen. A rigid metal plate is attached to the triaxial piston and a dial gage rests on the top of this deflection indicator. The dial gage is attached to the top plate of the triaxial cell so that movements of the entire cell will not be recorded on the dial gage.

The load applied to the triaxial specimen was controlled by adjusting the pressure regulator. For the first few load cycles, the applied pressure

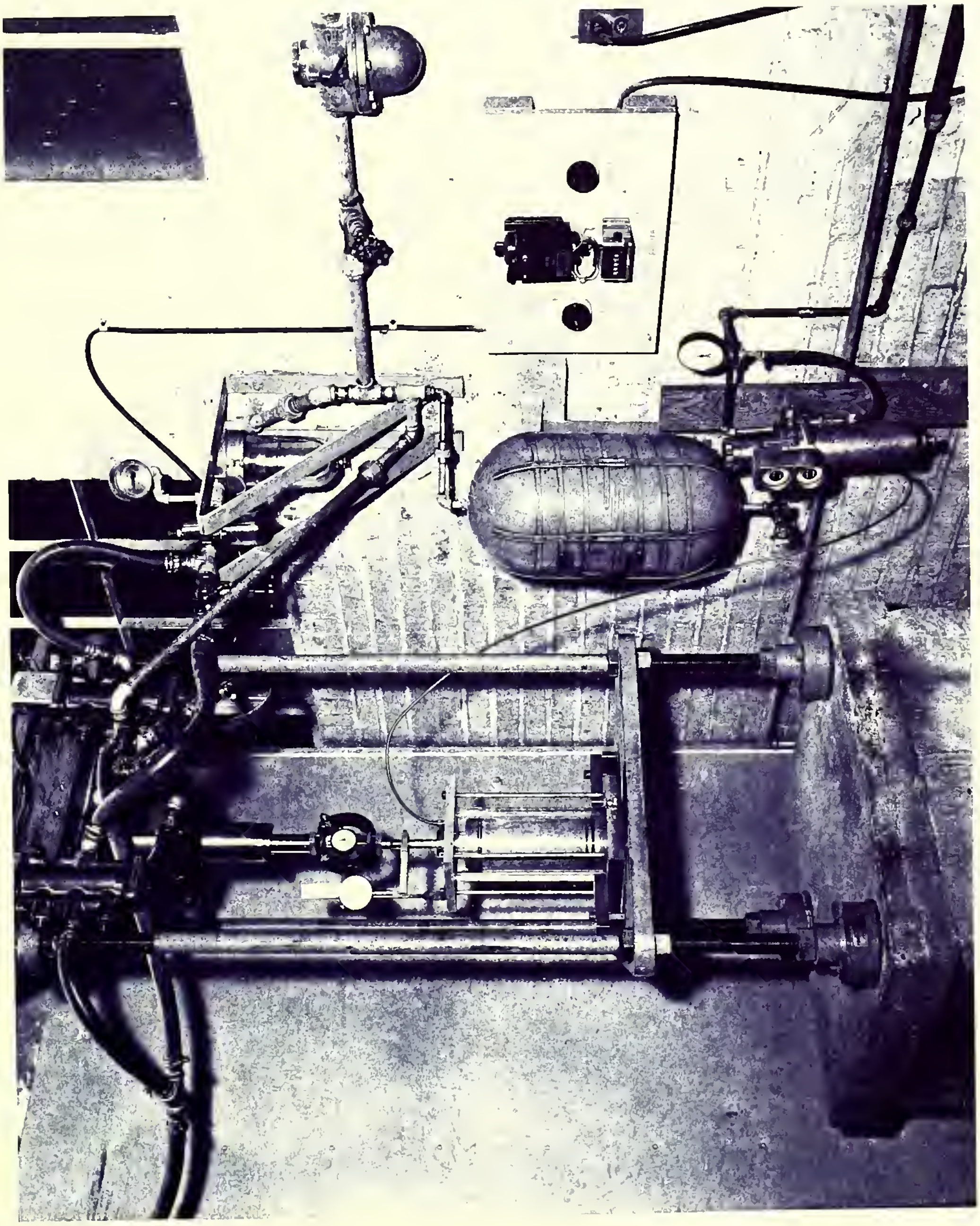

FIG. 3 EQUIPMENT USED FOR REPEATED LOAD TESTS

was regulated by adjusting the reduced line pressure to 14.5 psi. This corresponded to a specimen deviator stress of between 54 and 56 psi. After the machine was started, fine adjustments were made by referring to the proving ring dial indicator. The time interval between successive load applications and the duration of the applied load were controlled by means of a timer which furnished electrical impulses to the solenoid valve. These impulses opened the solenoid valve, thus allowing air to enter into and exhaust from the pressure cylinder.

A trace of applied load versus time for two typical load cycles is shown in Figure 4. These traces were obtained using an oscillograph recorder, amplifier, and strain gages. The proving ring, previously described, was fitted with four SR-4 strain gages which were connected to the amplifier. The signal was transferred from the amplifier to a permanent trace on the graph paper located in the recorder. The strain gages were calibrated, as was the proving ring, prior to beginning the testing program. It will be noted that there is no observed impact. As can be seen, the load increased rapidly at a decreasing rate until full load was reached. This decrease in rate of load application was due to a drop in line pressure as the compressed air flowed into the pressure cylinder. Due to these mechanical limitations, 0.12 second was required to attain full load. The exhaust of pressure took approximately the same length of time. The full load was maintained for only .04 second making a total load cycle of 0.28 second. Studies at the Stockton test track (10) have shown that a wheel distributes vertical stress over a linear distance of about four feet at a depth of about two feet. The traffic at the AASHO Road Test was maintained at a constant speed of 30 miles per hour. A vehicle travelling at this speed would therefore load an increment of area for about 0.1 second. From the

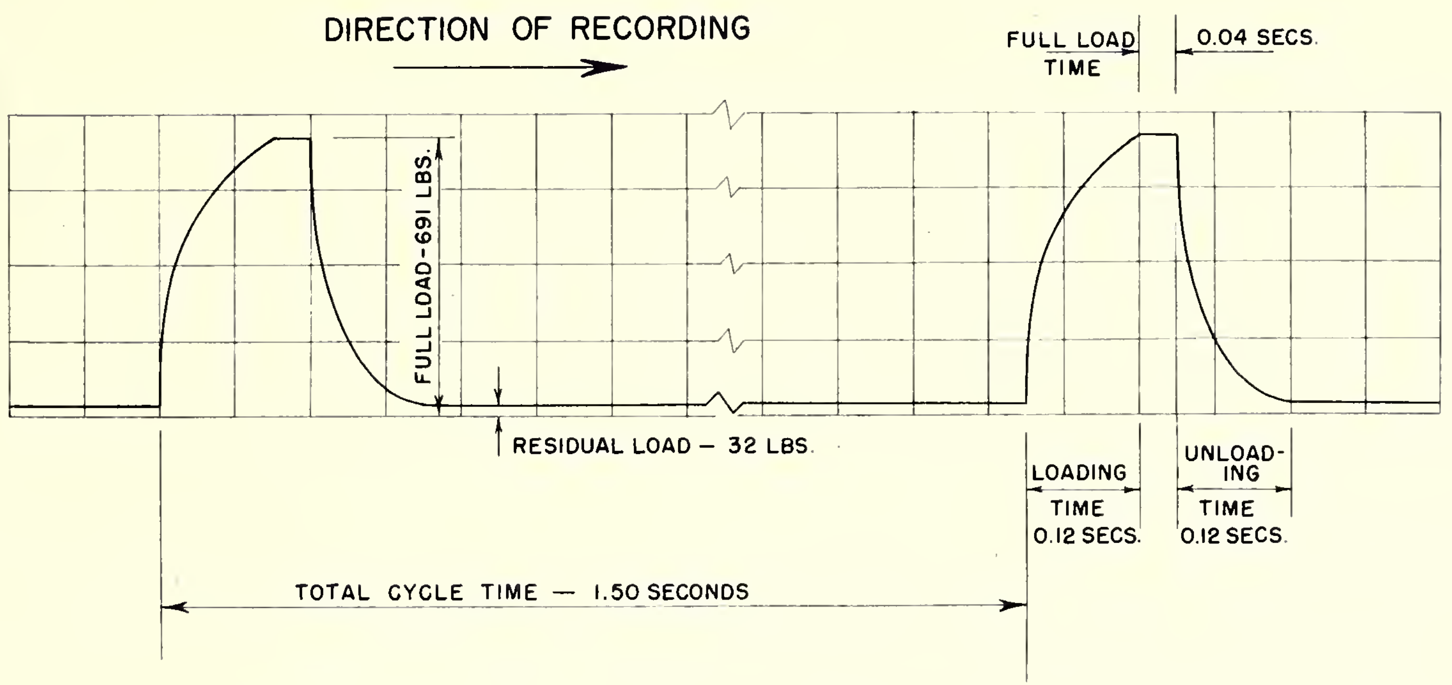

FIG. 4 TYPICAL LOAD — TIME TRACE FOR TWO LOAD CYCLES

above, it will be noted that each load cycle was active for a time period about three times this load duration; however, it was impossible to decrease this load time with the equipment used. The time interval from full load to full load was maintained at about 1.5 seconds. Therefore, the specimen could be loaded 40 times per minute or 2400 times per hour. As most of the specimens were subjected to 100,000 load cycles, it took slightly less than two days to perform each test.

The triaxial cell used in these tests was designed by the author and is pictured in Figure 3. The sleeve which surrounds the loading piston was made of Teflon, a relatively frictionless plastic, which reduced the side friction considerably. The cell was made to test specimens four inches in diameter and eight inches in length. The solid base pedestal was made so it could be removed and could be replaced by a commercial spiral grooved base pedestal. This feature was incorporated as it was desired to saturate some of the porous specimens after the specimen had been placed in the triaxial cell. However, the great majority of specimens were saturated prior to being placed in the triaxial cell, in which case a solid cap and pedestal were used. The base plate contained valves for the entrance and exit of water used in saturating specimens in place and a valve for draining the water used as the confining medium for the specimens.

When a specimen was ready for testing, it was transferred to the base pedestal of the triaxial cell and positioned. During this operation, the specimen was supported by a thin round metal plate which has been previously described. This round plate was grooved to receive an O-ring which provided an additional seal against leakage. Vacuum grease was applied to the pedestal and top loading cap which was positioned on top of the specimen. Two rubber membranes were then placed around the triaxial specimen. Two

membranes were used to insure against leakage during the testing period. O-rings were then placed around the top and bottom supports of the specimen. Grooves were provided in the pedestal and top cap so that the O-rings would provide a good seal. The Lucite cylinder and top plate of the triaxial cell were then positioned and the four bolts secured so that the cell would not leak. The cell was then filled with water to within one half inch of the top and the compressed air line was attached. The pressure was increased to 15 psi and allowed to remain about 30 minutes prior to commencement of the test. An initial deflection dial reading was taken prior to beginning the test. The appropriate valves were next opened, the load counter was set to zero, and the test was begun.

During the first few hundred cycles, deflection readings were taken fairly often. Later, the interval of recording readings was increased, the increase in interval depending on the progress of sample failure. Two readings were taken during each cycle studied. One was the total deformation reading from the beginning of test to the position of the top of the specimen under full load at this observed load cycle. The second was the distance the specimen rebounded from full load to residual load. In this position, the repeated deviator stress had been removed but the pressure caused by the weight of the proving ring and pressure cylinder piston was still present, thus the reason for calling the off load a residual load.

When the specimen had been subjected to 100,000 load cycles or had deflected a total of about one half inch, whichever occurred first, the test was terminated and the pressure cylinder piston was raised. The drain valve was opened and the water surrounding the specimen was removed. The lateral pressure was released and the triaxial cell was disassembled. The O-rings around the pedestal of the base plate were moved up over the O-ring

that was attached to the small round metal disk on which the specimen rested. The entire specimen, disk, and membranes were now removed without fear of loss of moisture.

RESULTS

Compaction tests were first performed to determine the compactive effort required to obtain the desired level of density for each gradation of the gravel and crushed stone material. The results of these tests are presented in Figures 5 and 6. These tests (see section entitled Compaction Procedure) were performed using a constant moisture content for each material. As can be seen, a straight line relationship resulted when the results were plotted on semilog paper. The horizontal dashed line which has been drawn on each figure indicates the level of dry density used in the repeated loading tests.

The deflection histories of the individual specimens are presented in Figures A-1 through A-6 in the Appendix. Figures A-1 through A-3 pertain to the gravel specimens, each figure containing the results for a specific gradation and Figures A-4 through A-6 contain the results of the crushed stone tests grouped in the same manner. The table which accompanies each figure contains information relative to computed dry density and degree of saturation for each test specimen. The reported saturation levels are average values for the entire specimen height and are based on apparent specific gravities. The curves which are presented in these six figures show the progressive deformation of the specimens with increasing repetitions of stress application. The plotted values are total cumulative deflection (difference from initial height of the specimen to the height measured under full load for the cycle plotted). The code which identifies each group of specimens according to its basic material and the gradation of that material is as

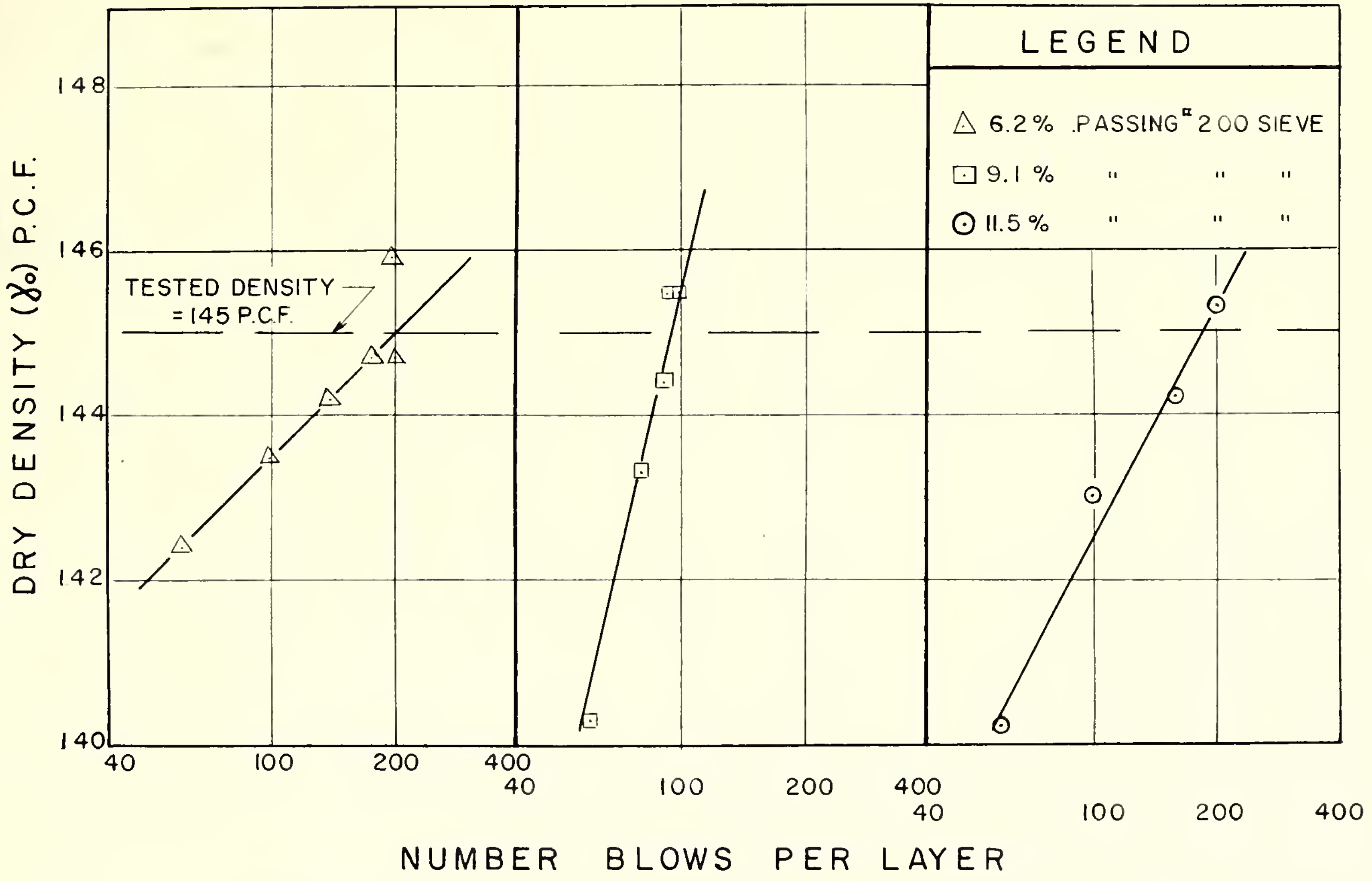

FIG. 5 VARIATION IN DENSITY WITH COMPACTIVE EFFORT — GRAVEL

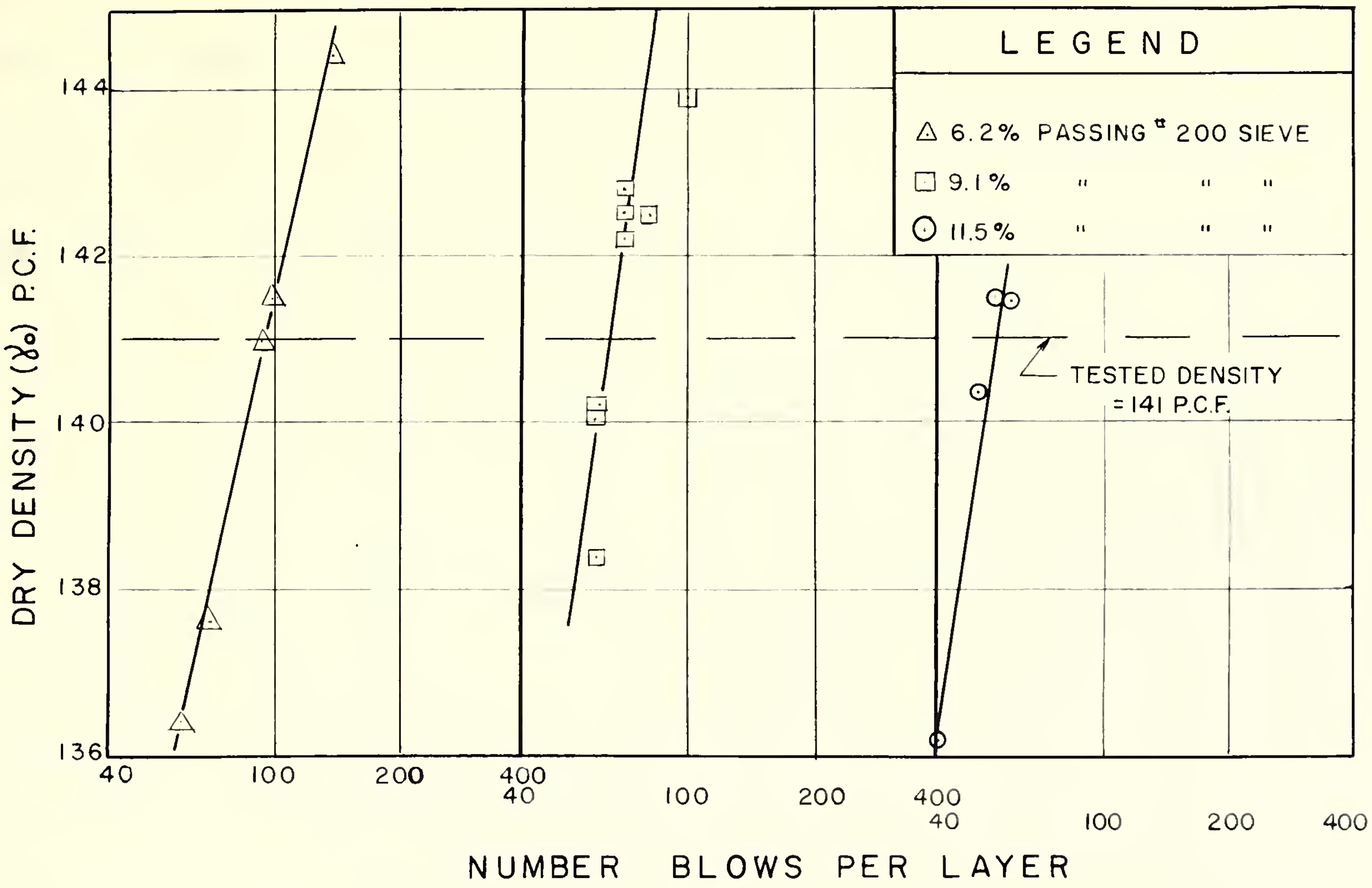

FIG. 6 VARIATION IN DENSITY WITH COMPACTIVE EFFORT – CRUSHED STONE

follows. The first figure designates either gravel (G) or crushed stone (S) material; the number which follows designates the number assigned to the specimen in that particular test group and the last figure designates the amount (in percent) of material passing a number 200 mesh sieve; A = 6.2 percent, B = 9.1 percent and C = 11.5 percent.

The rebound histories from full load to residual load for the same specimens are shown in Figures A-7 through A-12 and are grouped in the same manner as those previously mentioned. The data plotted in Figures A-1 through A-12 have been summarized for convenience of comparison in Figures 7 through 10.

Figures 7 and 8 summarize the deflection histories of the gravel and crushed stone mixtures. These curves represent averages of the original test results grouped according to the gradation and the degree of saturation of the test specimens. Figures 9 and 10 contain summaries of the curves shown in Figures A-7 through A-12. The rebound data are grouped in the same manner as the deflection data .

Figures 11 and 12 show a plot of the total deflection of the triaxial specimens at the 1000th load cycle with degree of saturation. Figure 11 represents this data for the gravel specimens and Figure 12 for the crushed stone specimens. The relationship between the rebound at the 1000th load cycle and degree of saturation is shown in Figures 13 and 14.

Referring to the curves in Figures 7 and 8, the trend that is most readily apparent is the decreased resistance to deformation of both the gravel and crushed stone specimens as the degree of saturation was increased. This variation in strength is further illustrated in Figures 11 and 12. An arbitrary level of 1000 load cycles was chosen for comparison as some

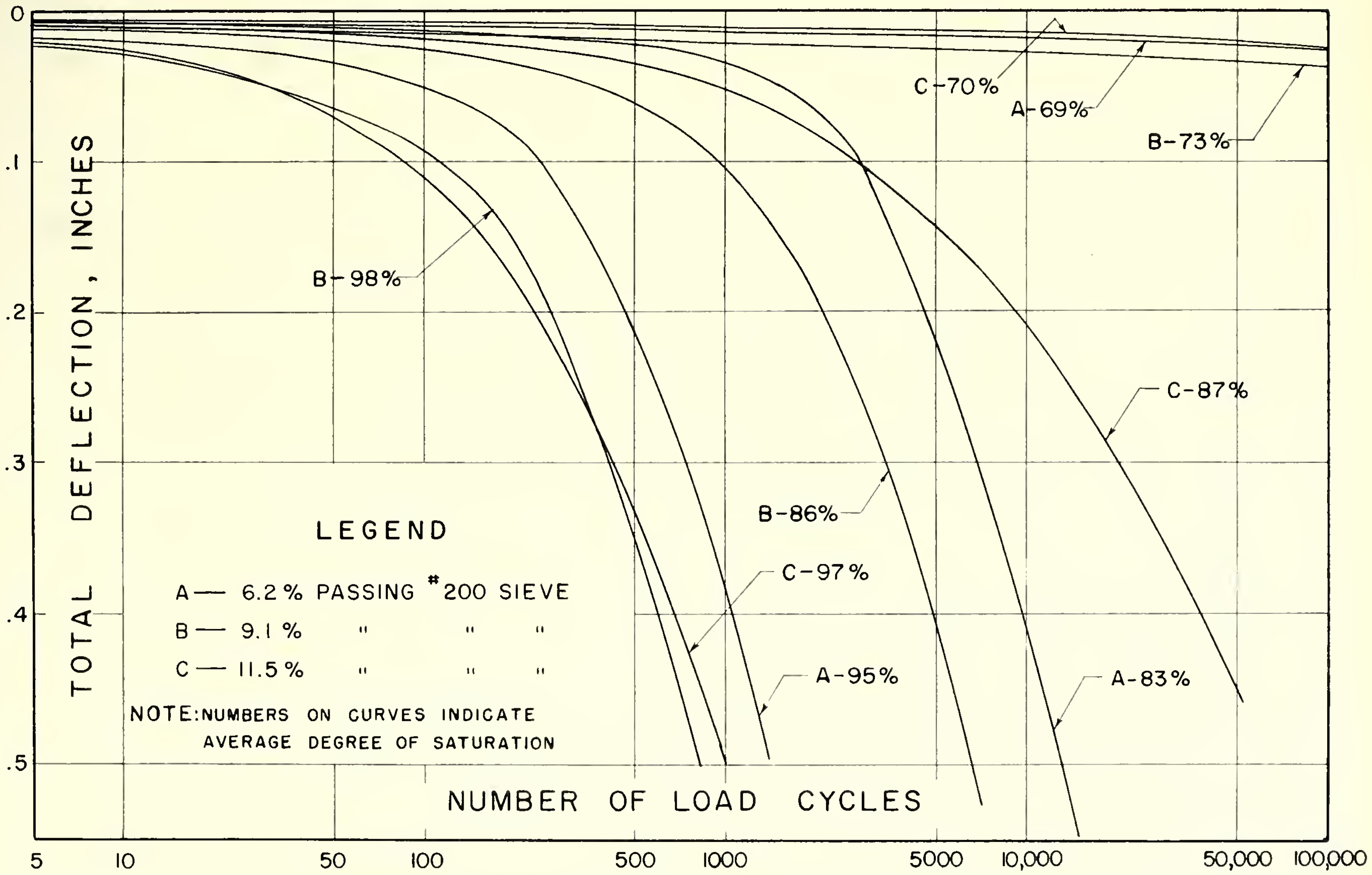

FIG. 7 DEFLECTION HISTORY OF GRAVEL SPECIMENS

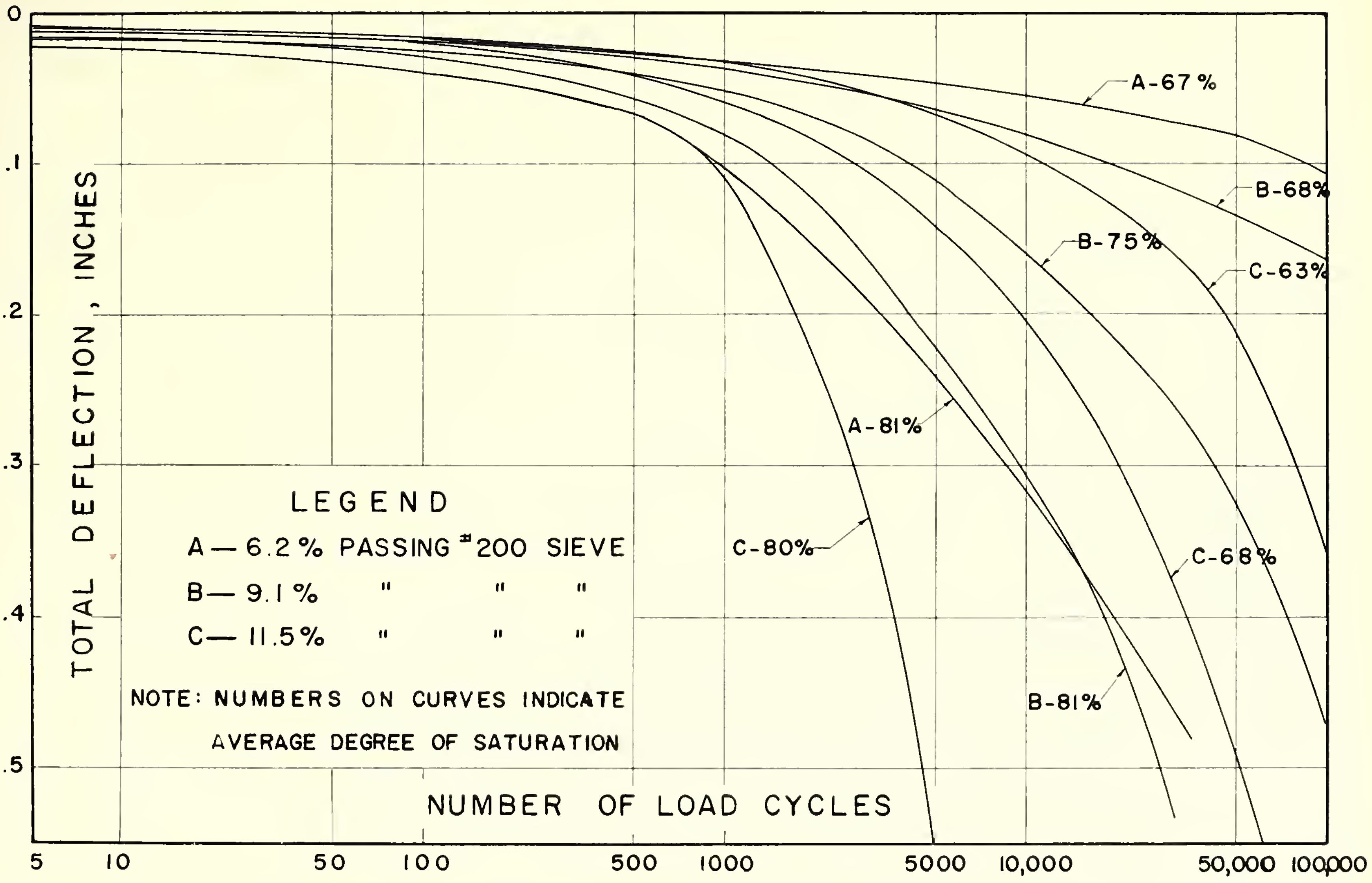

FIG. 8 DEFLECTION HISTORY OF CRUSHED STONE SPECIMENS

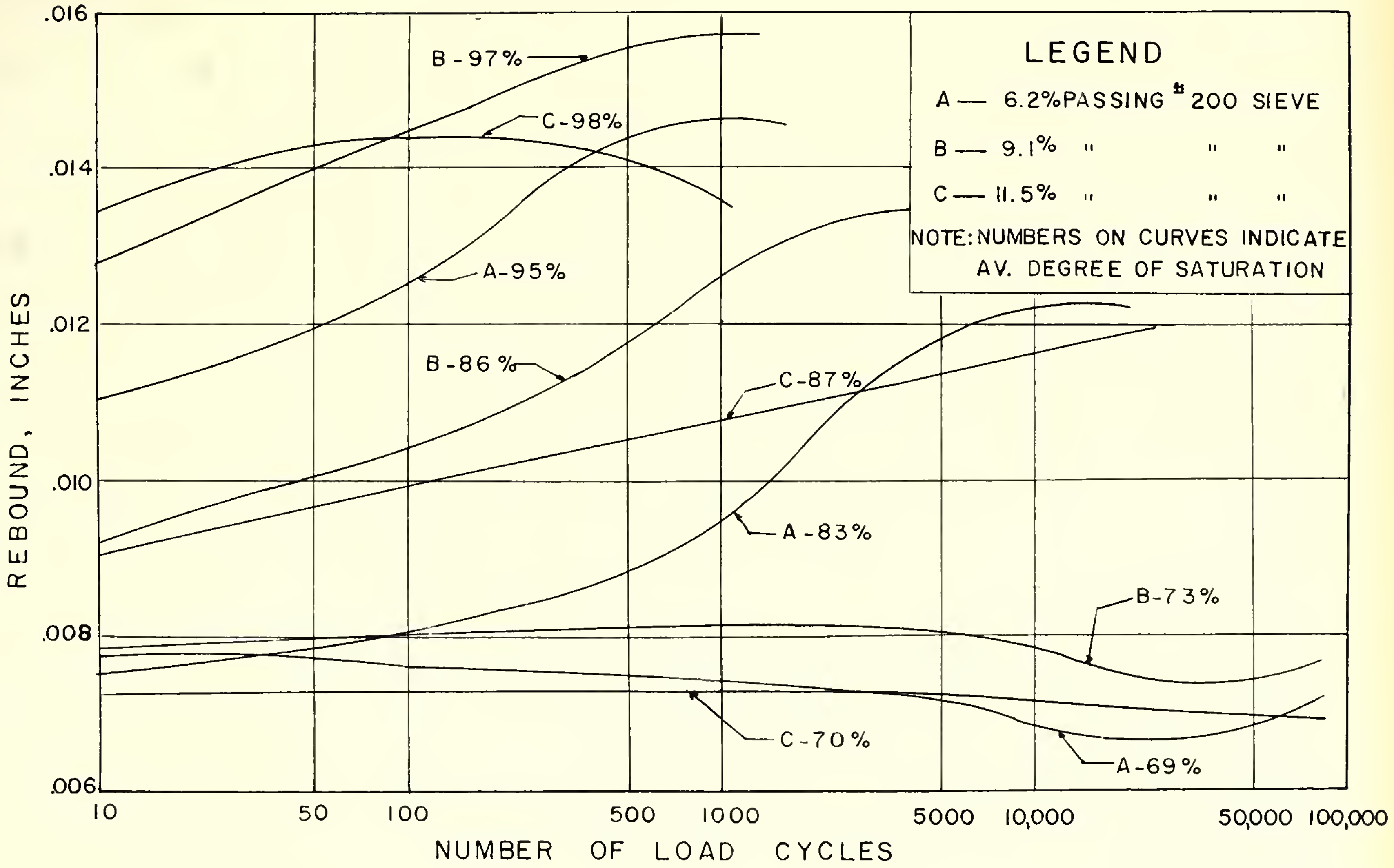

FIG. 9 REBOUND HISTORY OF GRAVEL SPECIMENS

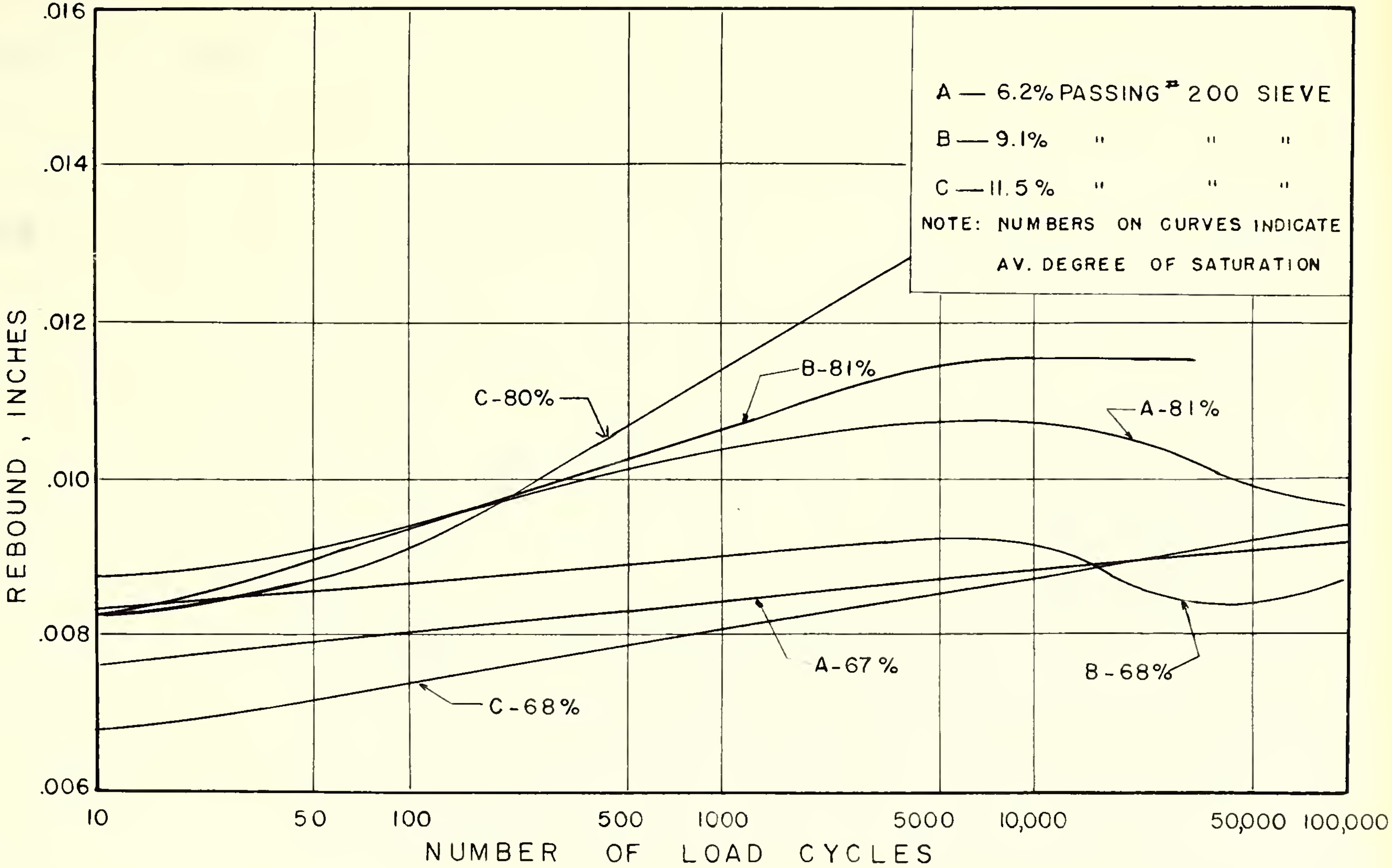

FIG. 10 REBOUND HISTORY OF CRUSHED STONE SPECIMENS

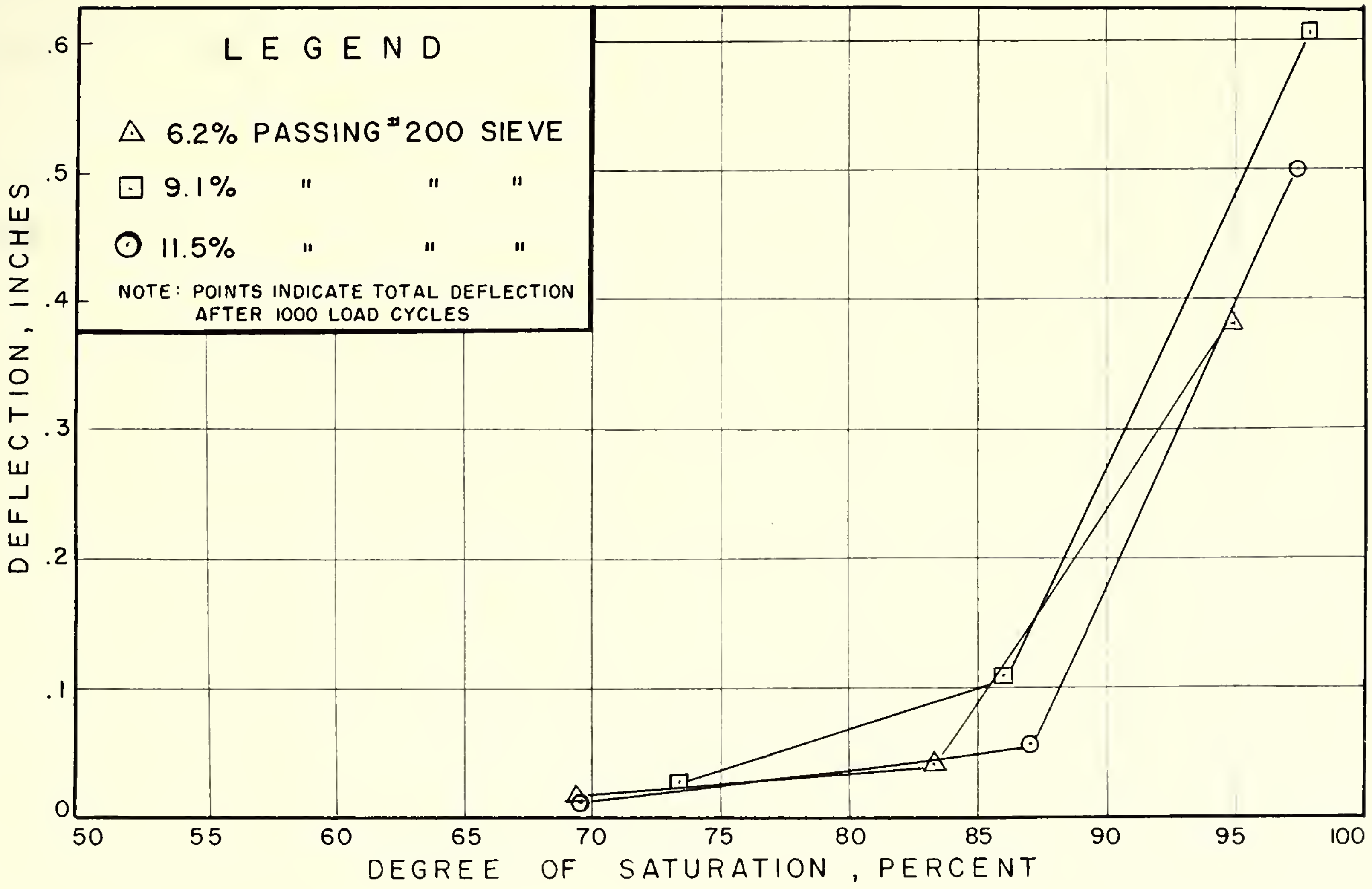

FIG. 11 VARIATION OF DEFLECTION WITH SATURATION LEVEL — GRAVEL

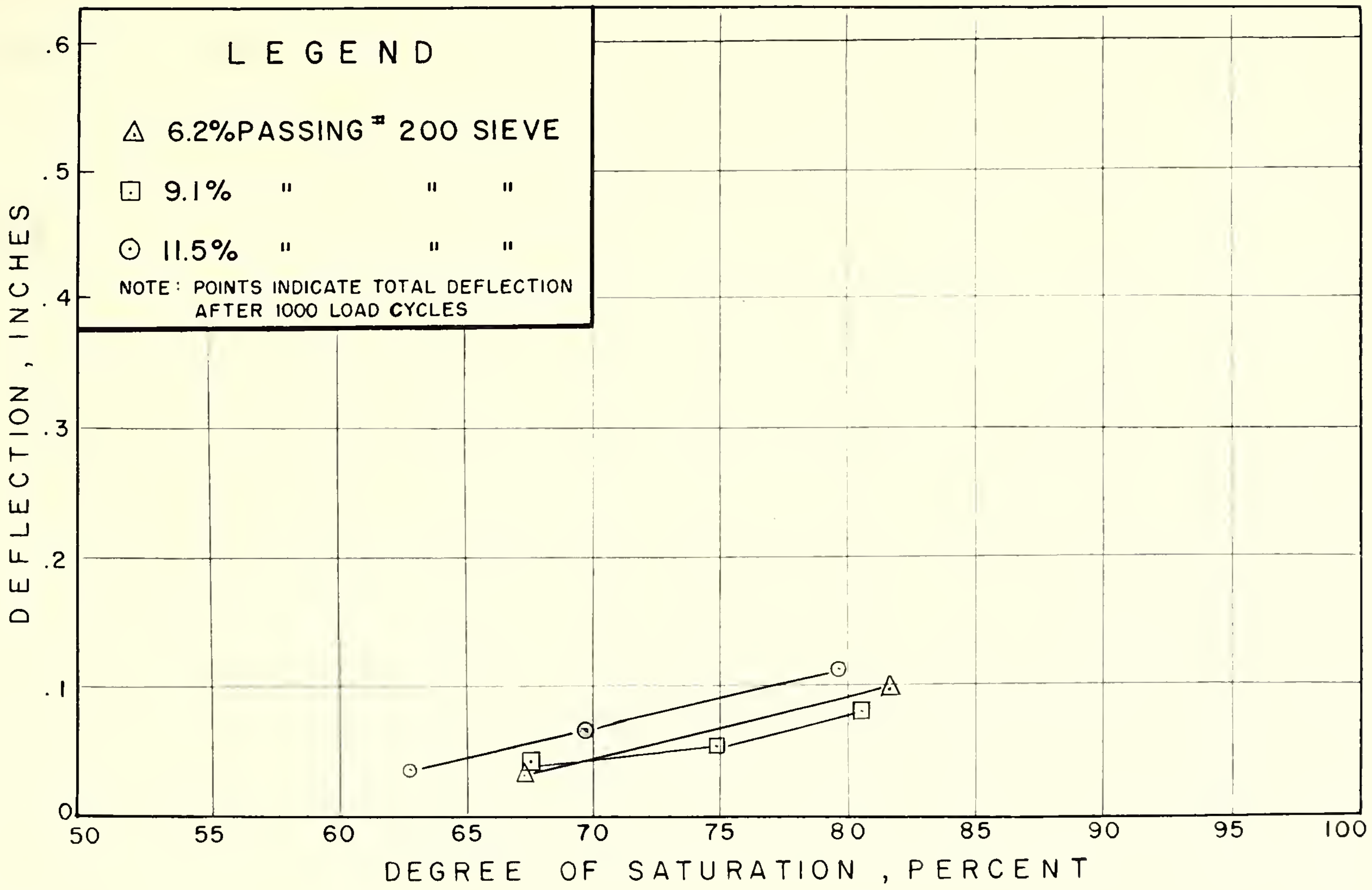

FIG.12 VARIATION OF DEFLECTION WITH SATURATION LEVEL — CRUSHED STONE

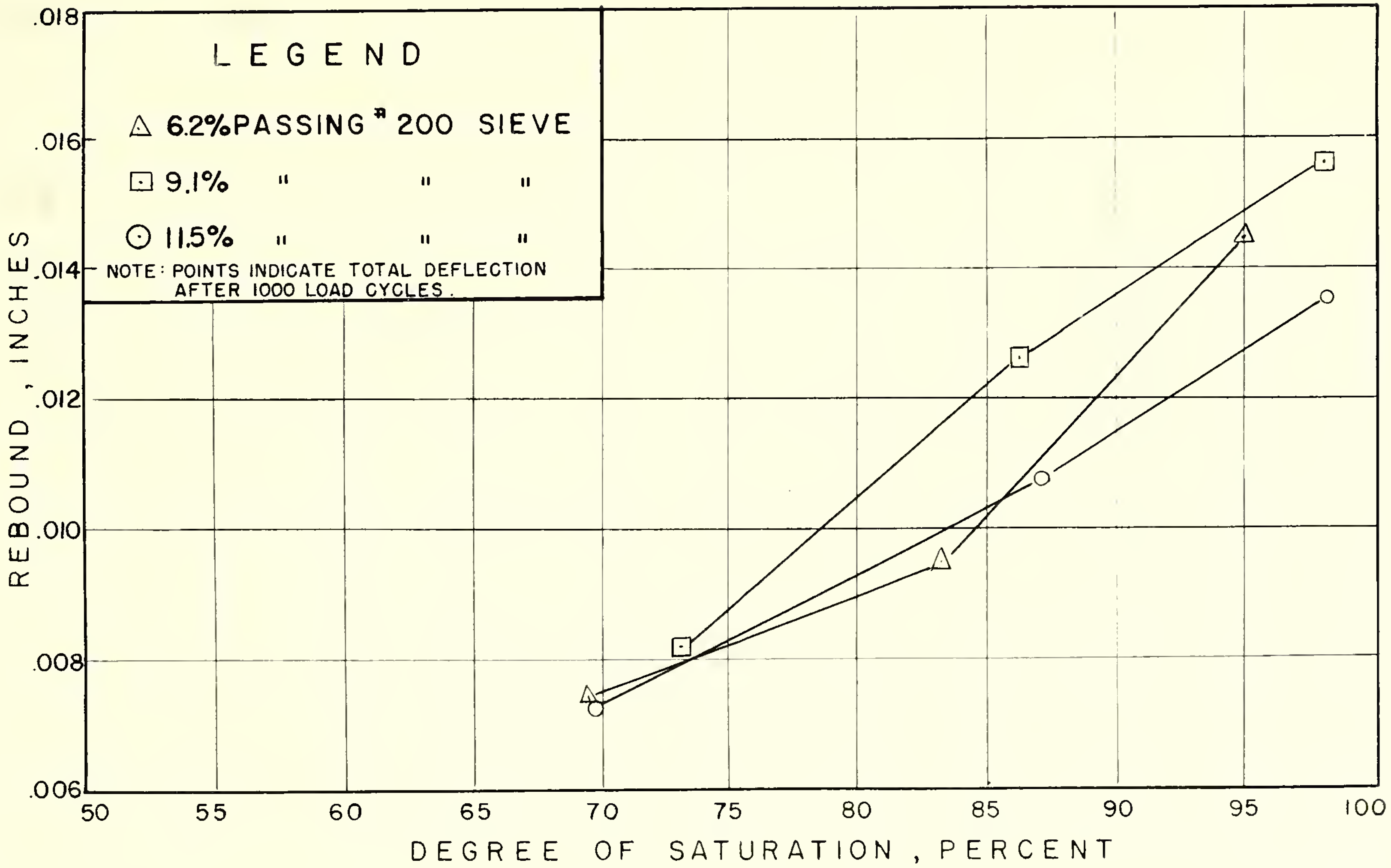

FIG. 13 VARIATION OF REBOUND WITH SATURATION LEVEL — GRAVEL

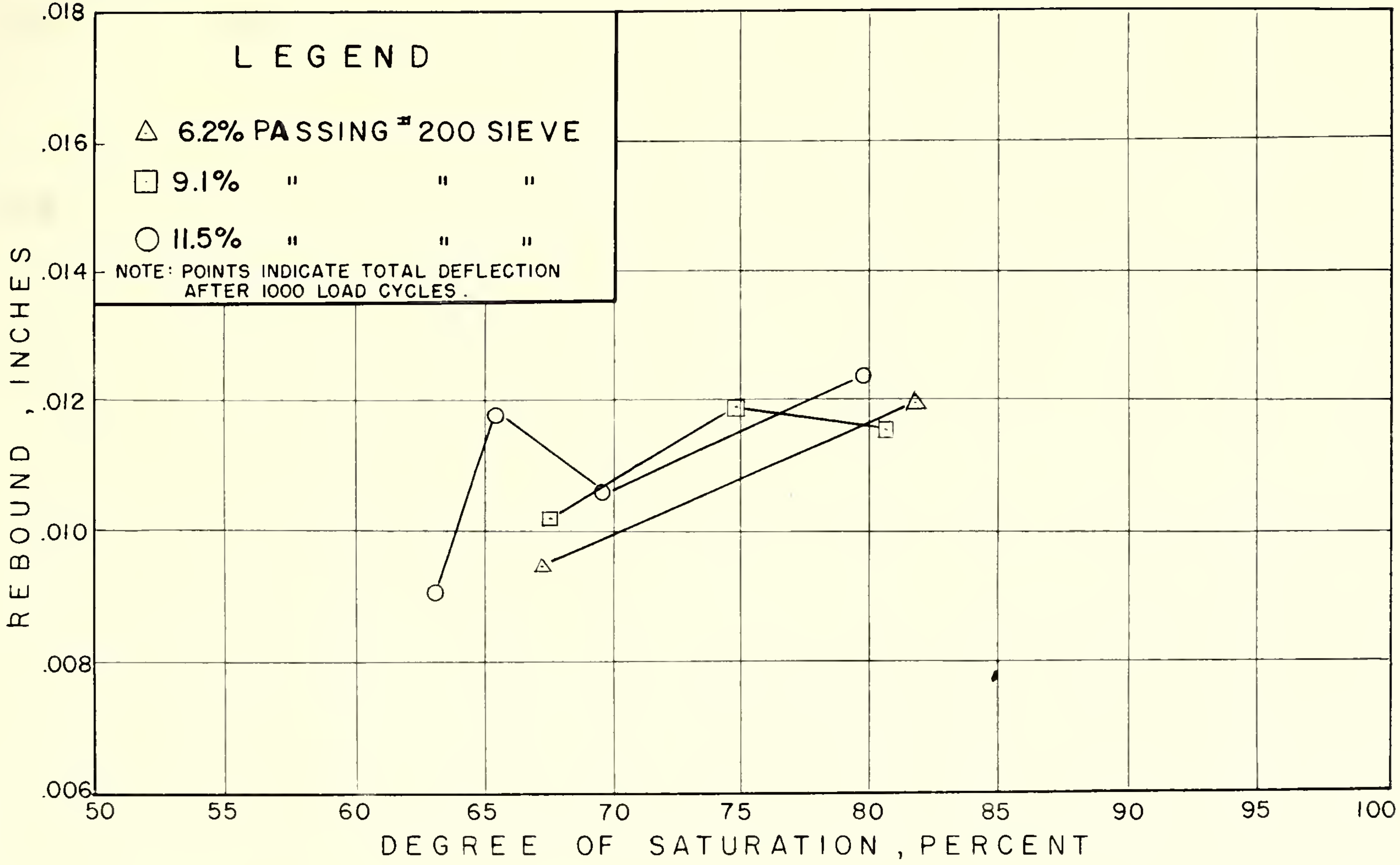

FIG. 14 VARIATION OF REBOUND WITH SATURATION LEVEL — CRUSHED STONE

of the specimens with high degrees of saturation deflected as much as seven percent of their original height at this point. Data beyond 1000 cycles was considered to be unrealistic due to distortion in the shape of the specimen. Since the average cross sectional area increases with axial strain, the deviator stress decreases significantly and data at high repetitions of load become unrealistic. Therefore, there is an increasingly significant error in all the test results as the specimens continue to deform.

To maintain a constant repeated deviator stress it becomes necessary to increase the axial load, as the average cross sectional area increases with number of load applications. In the triaxial test, the average cross sectional area can be calculated by the following formula:

$$A_a = A_o \left[\frac{1 + \Delta V/V_o}{1 - \Delta L/L_o} \right]$$

Where

A_a = average cross sectional area

A_o = original cross sectional area

ΔV = change in volume of specimen

V_o = original volume of specimen

ΔL = change in length of specimen

L_o = original length of specimen

In an undrained test made on a completely saturated specimen, in which there is no change in volume with axial strain, the value A_a can be

calculated at any desired load cycle directly from the measurement of the change in height of the specimen at that specified load cycle. Since the degree of saturation of most of the test specimens was below 100 percent and the change in volume was not measured, the revised cross sectional area could not be obtained.

Assuming that the volume of a specimen remains constant, a decrease in deviator stress of three percent results from a drop in total deviator stress from 55 psi to 53.35 psi. This decrease in pressure on the specimen results in total deformation and rebound values which are low. If the axial deviator load remains constant, a three percent decrease in deviator stress results from an axial strain of three percent (total deflection of the eight inch high specimen equal to 0.24 inch). At seven percent strain, corresponding to a total axial deformation of 0.56 inch, the accompanying decrease in deviator stress is 3.85 psi. Therefore, in all the subsequent analysis, it should be kept in mind that fairly large inherent errors in the trends may exist at high strain values.

In addition to errors that may be introduced at large strains, inherent errors exist at low strains. As was discussed in previous sections, the process of leveling the top of the specimen in the compaction mold was difficult due to the granular nature of the soil. As a result, a small variation in density near the top of the specimen inevitably resulted. The magnitude of the variation in density depended on the size and amount of selective material that was used in this region to fill the voids left after the large particles which protruded above the rim of the compaction mold had been removed. It is believed that this variation in density near the top of the specimen was reflected in the deflection of the specimen during the first few load applications during which time the material in

this region consolidated to some degree. However, after the deflection of the specimen had reached about 0.03 inch, the effect of the initial deflection in the top zone of the specimen compared to the deflection of the entire specimen was negligible.

Further, for those specimens tested at a low degree of saturation, the difference in deflection between the three gradations at 50 and even 100 load cycles was almost negligible. This slight difference in deflection was most apparent in the crushed stone specimens (Figure 8). It is therefore felt that comparisons of the data based on 1000 load cycles illustrate the significant trends in the most realistic manner possible.

As has been previously stated, it was desired to maintain the dry density of all gravel specimens at 145 pcf and the densities of the crushed stone material at 141 pcf. The test results of specimens in which the dry density varied from the specified values by more than two percent were omitted from the analysis. It will be noted in Figures A-1 through A-6 that the dry density of the specimens varied by not more than 1.8 percent for the gravel mixtures and 1.2 percent for the crushed stone mixtures and the degree of saturation varied a small amount from the desired levels.

It was found that a slight variation in the degree of saturation resulted in a great difference in the stability of the specimens. The test results of the specimens for each gradation were therefore grouped according to their average degrees of saturation approximating each saturation level. For example, tests S-2-A and S-3-A had degrees of saturation of 66.5 percent and 67.8 percent, respectively. The deflection curves and degrees of saturation for these two tests were averaged; the average deflection history is that which is reported in Figure 8 as the curve identified A-67 percent. It will be noted that the other two gradations of crushed

stone in the 70 percent saturation range had average degrees of saturation of 68 percent. Two specimens tested using a gradation of 11.5 percent passing the number 200 sieve had degrees of saturation far below this range and are reported in Figure 8 as having an average degree of saturation of 63 percent.

It was difficult to maintain the specimens of crushed stone in the range of the three desired levels of saturation. This difficulty was due to the relatively high permeability of the crushed stone specimens. It will be seen from Figure 17 that for the gradations and densities used in the AASHO Road Test, the coefficient of permeability of the gravel was found to be about 0.01 foot per day and that of the crushed stone about 8.5 feet per day. Upon removal of the crushed stone specimens from the saturation mold, they drained readily and lost most of the water added by the saturation process before they could be positioned in the triaxial cell. Even the specimens of crushed stone compacted at 70 percent saturation drained to some extent and as can be seen in Figure A-6, the degree of saturation of one of the specimens was reduced to 62 percent.

As it was desired to obtain test results for the crushed stone materials at a degree of saturation close to 100 percent, the method of saturation was modified for these tests because of the drainage problems discussed above. The solid loading cap and base of the triaxial cell were replaced by spiral grooved caps with drainage outlets and porous stones were placed on top and bottom of the specimen. The specimen was compacted as before to an initial degree of saturation of 70 percent. After the triaxial cell was assembled with the specimen in place, a vacuum was applied to the top of the specimen and water was allowed to enter from the bottom. By this method, the degree of saturation was raised to between 85 percent and 95

percent. However, the results on specimens tested in this range of saturation were found to be unsatisfactory as the water in the specimen migrated, due to the force of gravity, toward the bottom of the specimen during the insuing repeated loading test period. This migration of water was evident from the resulting uneven bulge of the specimens as deformation increased. In all cases, the bottom of the specimens bulged whereas the upper zone changed shape only slightly. Upon examination of the failed specimens, the upper portion of most of these specimens appeared to be relatively dry and undisturbed whereas the lower portion was highly disturbed and comparatively wet. A few of these specimens were split into three sections for moisture content determinations. The degree of saturation of the top, middle and bottom portions was then calculated using the initial dry density for these determinations. The results of these calculations are shown in Figures A-5 and A-6. As these variations in the degree of saturation influenced the test results, these tests were not included in the comparative curves shown in Figure 12.

Since the shape of the failed specimens of crushed stone with a degree of saturation equal to or less than about 80 percent was symmetrical, it is felt that these data are valid. Evidently, the moisture in these specimens was held in place by capillary forces.

Satisfactory results were obtained when testing the gravel due primarily to the highly impervious nature of the material. It was possible to fabricate gravel specimens in the range of 85 and 100 percent saturation. No migration of water was noted in the gravel specimens during the test period. The saturation procedure described in the section on Compaction Procedure was found to be satisfactory in saturating these specimens to any moisture level desired. Upon removal of the specimens from the saturation mold, little if any drainage was observed.

A summary of the dry densities of the materials in their loosest and densest state and the relative densities of the materials at the specified field density levels is given in Table 3. The loosest state was obtained by gently placing the material in the compaction mold and the densest state was obtained by controlled vibratory compaction. The laboratory compactive efforts required to obtain the desired field dry densities are shown in Figure 15. The relative densities of the material at the specified levels of 145 pcf and 141 pcf for the gravel and crushed stone are indicated adjacent to each point on the curves.

TABLE 3

SUMMARY OF RELATIVE DENSITY DATA

Material	Gradation % Passing #200 Sieve	Loose Density (pcf)	Field Density (pcf)	Dense Density (pcf)	Relative Density (%)
Gravel					
	6.2	126.5	145.0	145.5	97.6
	9.1	127.0	145.0	146.6	92.8
	11.5	126.7	145.0	145.8	96.3
Crushed Stone					
	6.2	113.3	141.0	141.6	98.2
	9.1	113.5	141.0	144.9	90.0
	11.5	118.9	141.0	146.3	83.8

In Figure 16 (a), curves are shown which relate the dry densities obtained with three different levels of compaction effort for each of the three gradations of each material. It is recognized that three points do not adequately define a curve; however, as the trend developed here follows the accepted patterns for data of this type, it is felt that the relationship is nearly correct. The data for the dry densities were obtained from

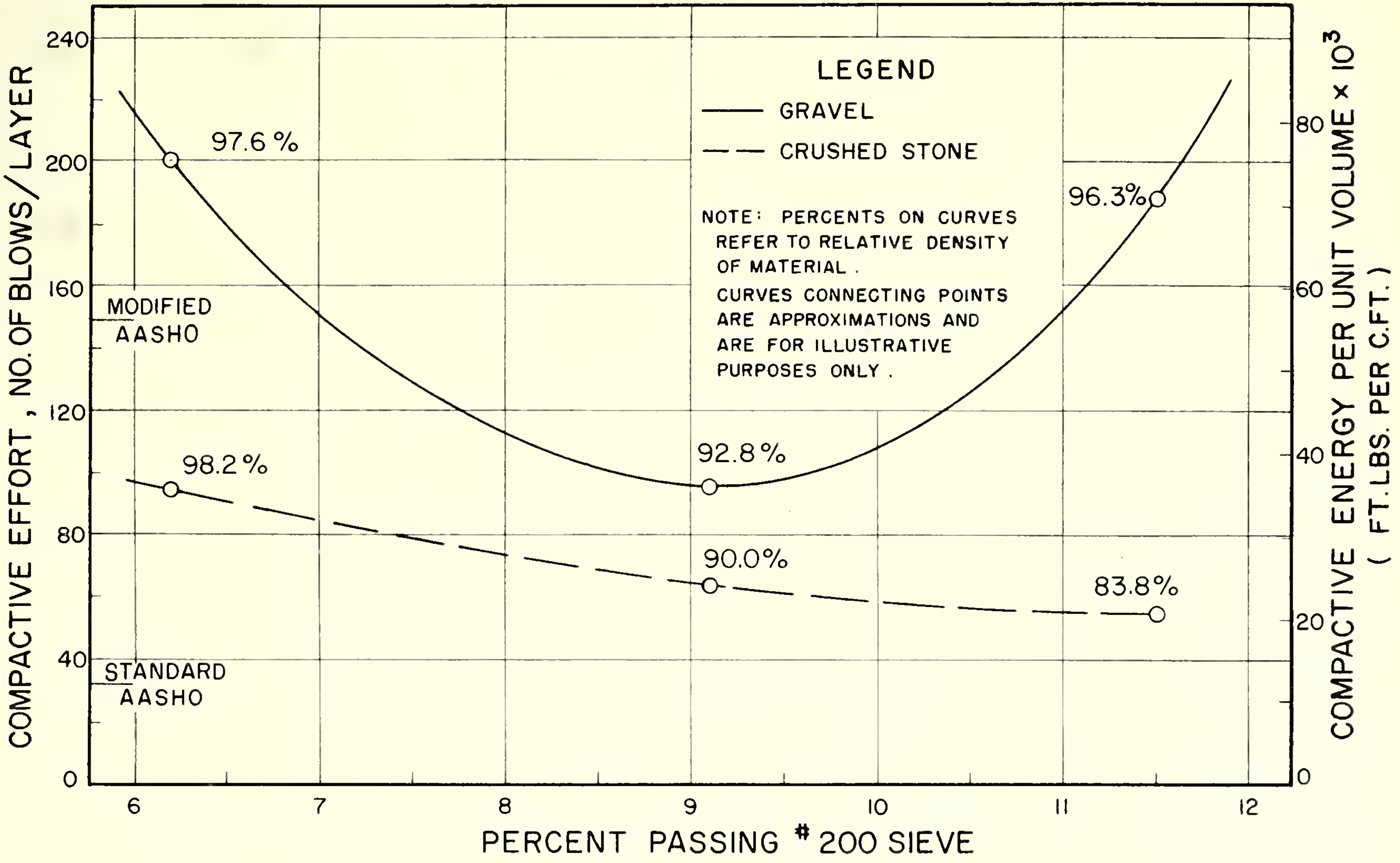

FIG. 15 COMPACTIVE EFFORT REQUIRED TO ACHIEVE FIELD DENSITIES

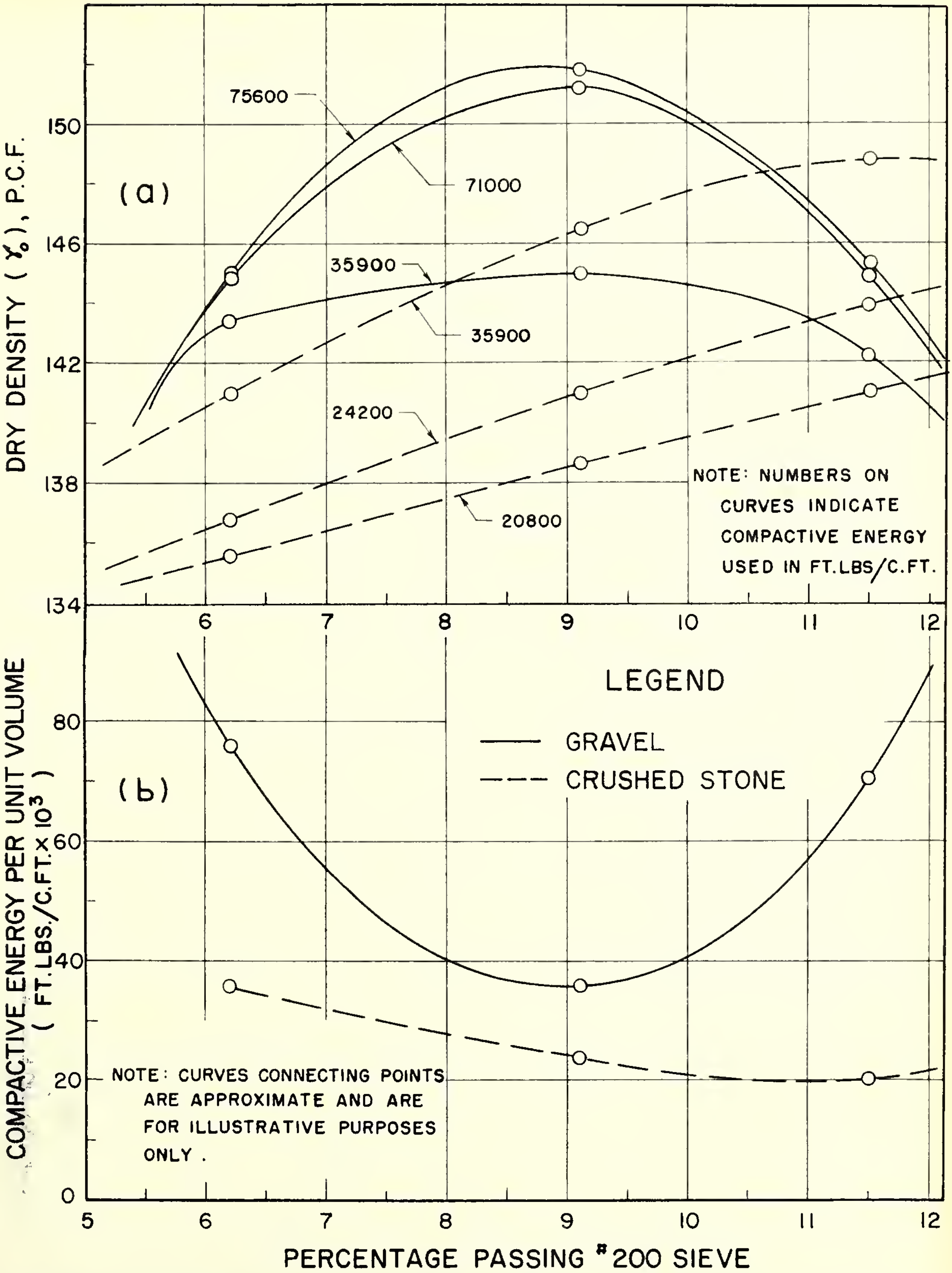

FIG.16(a) DENSITY OBTAINED AT VARIOUS COMPACTIVE EFFORTS
(b) COMPACTIVE EFFORT REQUIRED TO ACHIEVE FIELD DENSITIES

the curves shown in Figures 5 and 6. As some of the curves in Figures 5 and 6 do not contain data at the higher levels of compaction, these dry densities were found by extending the straight lines until the desired value of compactive effort was intersected. The compactive efforts used for the individual curves in Figure 16 (a) correspond to those used for each of the gradations of the two materials. For comparative purposes, the data presented in Figure 15 have been reproduced in Figure 16 (b).

It is known that compaction of granular material may cause aggregate degradation. Sieve analyses were made on representative test specimens to check their final gradation against the gradation of the mixtures prior to compaction. The results of the grain size distributions are tabulated in Table 4. It will be noted that there was an appreciable amount of aggregate breakdown. However, this aggregate breakdown appears to be uniform for all the gradations. For instance, the amount of material passing the number 200 sieve for the three gradations was originally 6.2, 9.1 and 11.5 percent. For the gravel, after compaction, these values had changed to 8.2, 11.5 and 13.8 percent whereas in the crushed stone material, the corresponding values were 7.8, 9.5 and 11.5 percent. This indicates that there was less breakdown due to compaction in the crushed stone than in the gravel. The tabulations are accompanied by the values for the gradations prior to compaction. Specimens which were selected for sieve analyses were those with essentially the same stress history. This selection was made in an effort to eliminate insofar as possible the variable of load applications. As the quantity of fines is of interest, it will be noted that all percentages passing the number 200 sieve remained in the same order of magnitude as before; therefore, the analysis of results are still valid except that the quantity of fines reported in all cases is slightly low.

TABLE 4

CHANGE IN GRAIN SIZE DISTRIBUTION DUE TO COMPACTION - PERCENT PASSING

Gravel

Sieve Size	6.2% Gradation G-1-A		9.1% Gradation G-3-B		11.5% Gradation G-3-C	
	Before Compaction	After Test	Before Compaction	After Test	Before Compaction	After Test
3/4"	100.0	100.0	100.0	100.0	100.0	100.0
1/2"	73.2	77.3	74.0	77.5	74.7	79.4
#4	47.4	52.4	49.0	53.7	50.4	55.1
#8	38.9	42.9	40.8	44.7	42.4	46.6
#16	32.9	34.8	35.0	37.1	36.7	39.3
#50	16.6	19.4	19.2	21.9	21.3	24.5
#100	10.0	12.1	12.8	15.0	15.1	17.7
#200	6.2	8.2	9.1	11.5	11.5	13.8

Crushed Stone

Sieve Size	6.2% Gradation S-5-A		9.1% Gradation S-2-B		11.5% Gradation S-3-C	
	Before Compaction	After Test	Before Compaction	After Test	Before Compaction	After Test
3/4"	100.0	100.0	100.0	100.0	100.0	100.0
1/2"	66.1	71.8	67.1	74.4	68.0	74.5
#4	47.0	49.1	48.6	51.8	50.0	53.4
#8	34.8	38.5	36.8	41.2	38.5	42.5
#16	25.3	24.0	27.6	25.2	29.5	31.8
#50	13.3	14.6	16.0	16.6	18.2	19.3
#100	9.4	11.0	12.2	12.8	14.5	15.7
#200	6.2	7.8	9.1	9.5	11.5	11.5

Figure 17 illustrates the variation in permeability of the two materials as dry density is increased. These permeability tests were performed on the gradations actually used in the test sections. Tests were made on each material at three levels of density.

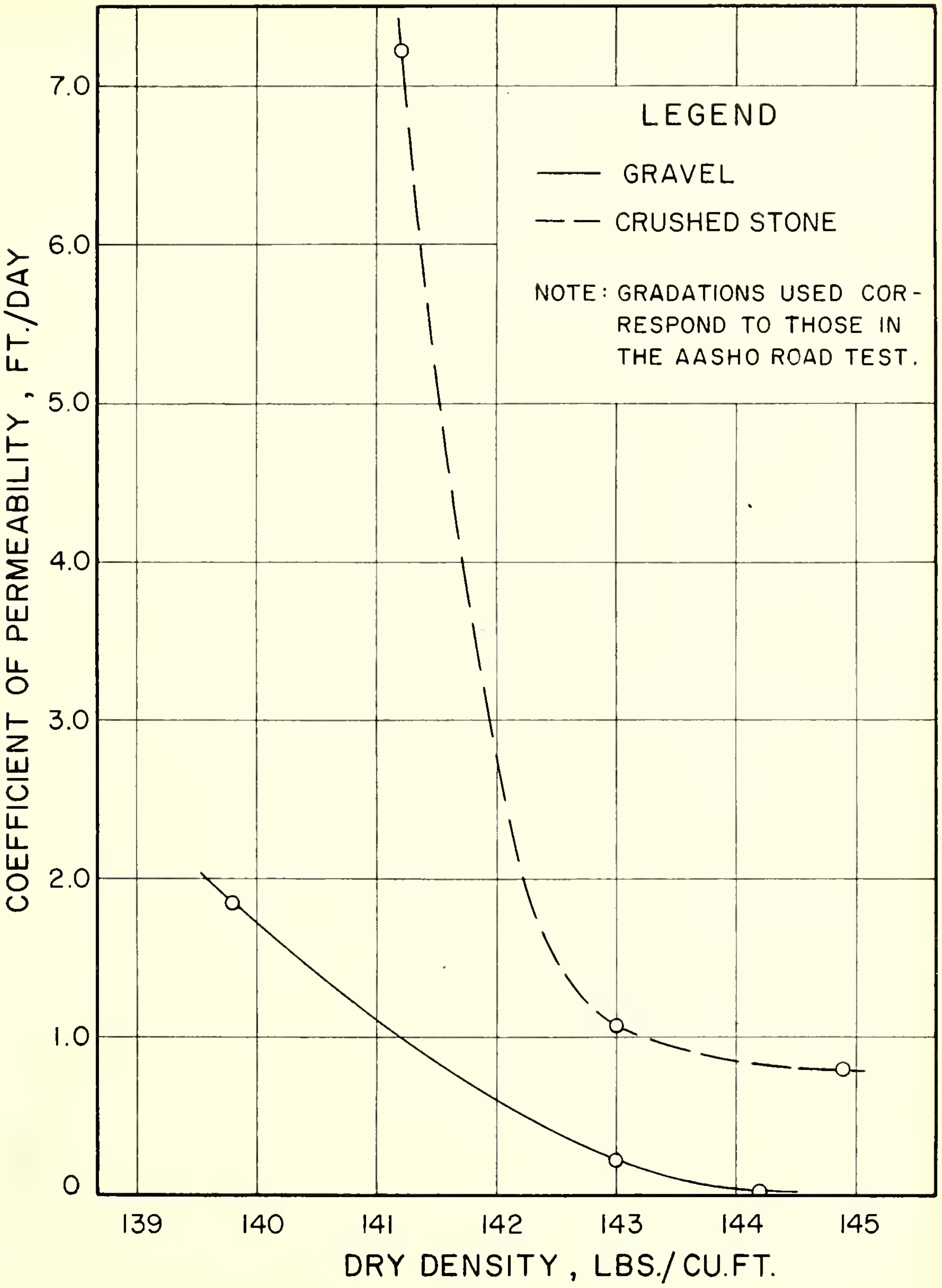

FIG. 17 VARIATION IN PERMEABILITY WITH DENSITY

DISCUSSION OF RESULTS

Compactive Energy and Relative Density

As a study of the two gradations of gravel and crushed stone tested at the dry densities used in the AASHO Road Test program was of primary interest, these two density levels were used in all the tests.

From Figure 15, it will be noted that the compactive effort required to attain the desired dry density for each gradation of the two materials varied considerably. The relative density of each of these materials compacted to the desired density is given by its corresponding point on each curve. As a rule, as the relative density of an aggregate mix increases, its performance is improved due to the decrease in its initial volume of voids. This was essentially borne out in the tests as will be discussed in subsequent paragraphs.

A base course material in the field is generally compacted with a uniform compactive effort for any given project. The base course receives the same number of passes of the compaction equipment regardless of slight changes in the gradation along the length of roadbed which inevitably results due to construction limitations. In other words, if one section of roadway contained a gradation which, when compacted by a specified number of passes, produced a higher dry density than the adjacent section due to a superior compactive ability of the material, the base course on the entire length of project would probably continue to be compacted until the materials with inferior compactive abilities met the compaction specifications. Therefore, the section which reached the specified dry density

first would now have a relative density exceeding that of the other sections.

Due to the above, equal compactive energies or equal relative densities are often specified in lieu of equal dry densities. The results of the repeated load tests in this study could have been compared on a basis of the densities produced by equal compactive efforts or they could have been compared on the basis of equal levels of relative density. However, both of these procedures would have involved an unwarranted amount of laboratory time. This, coupled with the fact that the principle purpose of this thesis was to evaluate the stability of the materials in the laboratory and to compare this evaluation with the field evaluation, led to the decision that the best density to use in the laboratory was that equal to the mean density of the materials in the field.

It can be seen in Figure 15 that the material with a gradation equal to that used in the AASHO Road Test required the least compactive energy of the three listed gradations to reach the desired dry density. It will also be noted that the relative density of each of these materials was the lowest in the group of three gradations tested for each material. It is also interesting to note the locations of standard AASHO and modified AASHO compactive efforts in relation to those used to obtain each point on the curves. One might conclude from the above observation that because of the low relative densities used, the two gradations picked for use in the Road Test were the worst possible choices. In order to discuss this premise more fully, a hypothesis based on data in Figure 16 is presented below.

It can be seen in Figure 16 (b), for the gravel material, that the compactive energy required to produce a dry density of 145 pcf was minimum when the amount of fines contained in the mix was 9.1 percent. At a very

low percent of fines, a great majority of large particles are in direct contact with each other and a large compactive effort is required to attain the desired dry density level. As more fines are added and the percent fines approaches the optimum point (for the gravel material this is close to nine percent) more of the fine material is deposited between the round surfaces of the larger particles. This "wedge" of moist material acts as a lubricant and aids the large particles in sliding past one another more easily. As the percent fines are increased, the compactive energy required to attain a dry density of 145 pcf decreases. Beyond about nine percent fines, the soil begins to hinder compaction as it chokes the voids and prohibits the large particles from moving into the void spaces. As a result, more compactive energy is required to attain the desired dry density.

It will be noted in Figure 16 (b) that the dashed curve for the crushed stone is much flatter. Unfortunately, data were not available to fully define the curve, but it appears that about 11 percent fines is the optimum point for this material. The same explanation for the shape of the curve for the gravel applies to the stone. A possible reason for the curve being so flat is that due to the angularity of the crushed stone and the resulting increased point to point contact of the larger particles, more fine material must be added before an effective wedge between the angular edges of the aggregate is developed. As the dry density of the stone is lower than that of the gravel material, the point of optimum fines is shifted to the right because of the larger void spaces between the aggregate grains. A greater quantity of fines, therefore, is required before the voids become choked.

The dry densities plotted in Figure 16 (a) have been extrapolated in part from data shown in Figures 5 and 6. The curves of Figure 16 (a) further explain the interaction of fines with the granular material of each mixture. For the gravel material with about six percent fines, a change in compactive energy from 35,900 ft. lbs. per cu. ft. to more than double that amount had almost no effect on the dry density of the mixture. This was probably due to the lack of adequate lubrication which would have been present if more fines were available. An increase in compactive effort, neglecting degradation which would inevitably result at these higher compactive energies, does not aid the larger particles in sliding into a closer network as the initial compaction has already produced close to 100 percent relative density for that mixture. As the percent fines is increased, this lubrication effect increases and additional compactive effort permits increases in density. Beyond the optimum point, which for gravel is about nine percent, the fines begin to build up on the interfaces between the large aggregate particles and begin to choke the voids. Due to this increase in concentration of fines, more compactive energy is transferred directly into the fine material. Thus, as the percent fines is increased, higher degrees of compaction of the fines result at lower compactive energy levels. Therefore, the curves for the various compactive efforts begin to converge as the percent of fines is increased.

The explanation for the shape of the crushed stone curves is the same as above except that the fines do not concentrate between the sharp, angular particles with increase in percent fines as fast as they do in the gravel materials. A final observation which is of interest is the shape of the crushed stone compactive energy curves in Figure 16 (a). Again, the curves are not **well** defined as data for percent fines greater than 11.5

percent are not available. However, it will be seen that the curve for a compactive energy equal to 35,900 ft. lbs. per cu. ft. begins to level out at about 11 percent fines and presumably will now begin to decrease in the same manner as the curves for the gravel. The curve representing a compactive effort of 20,800 ft. lbs. per cu. ft. has not begun to break at 11.5 percent fines and the intermediate curve appears to be increasing toward the right at a decreasing rate. This indicates that as the compactive energy increases, the optimum percent fines decreases. This is due to the lower values of the void ratio at these higher densities. In other words, as the void ratio decreases, the amount of fines required to choke the void spaces becomes less and, therefore, the resulting optimum percent fines for maximum dry density decreases as the compactive energy is increased. This same trend can be inferred from the curves plotted for the gravel gradations.

As far as compactive effort is concerned, it now becomes evident as to the advisability of picking the optimum percent fines for these two materials; about 9.1 percent for the gravel and about 11.5 percent for the crushed stone. From Figure 16 (a), if a dry density of 145 pcf is desired for the gravel material, this dry density can be attained using 9.1 percent fines with only about one half the compactive effort required to attain the same dry density using either the 6.2 percent or the 11.5 percent gradation. Further, for the gravel material if the higher compactive effort of 75,000 ft. lbs. per cu. ft. is used, one can achieve a dry density of about 152 pcf using the 9.1 percent gradation in contrast to about 145 pcf using either of the other two gradations. The above reasoning justifies the selection of the gradations used in the AASHO Road Test.

Regardless of the dry density level chosen for this laboratory study, Figure 16 (a) shows that less compactive effort would be required to attain the desired density level using about 9.1 percent fines for the gravel and about 11.5 percent for the crushed stone than for either of the other two gradations. Relative density varies with compactive effort, and as previously discussed, the performance of both the gravel and crushed stone should correlate closely with their relative densities. Therefore, from the standpoint of the relative density values shown in Figure 15, the gravel should become increasingly stable as the amount of fines is varied from 9.1 to 11.5 and finally to 6.2 percent (in terms of the coding system B, C and A) and those of crushed stone in the order of 11.5, 9.1 and 6.2 percent or C, B and A. Reference to this conclusion will be made throughout the remaining discussions.

Performance of Gravel

Referring to the results of the gravel mixtures shown in Figure 7, at low degrees of saturation the variation in stability of the three different gradations is fairly small. It will be noted that the difference in total deflection between samples A and C at 100,000 load cycles is less than 0.004 inch. As the relative density of these two gradations is 97.6 percent and 96.3 percent respectively, only a slight difference in total deflection is to be expected. Specimens with grading B are definitely less stable than those with the other two gradations due to their lower values of relative density, 92.8 percent. At a low degree of saturation, there is a relatively large quantity of air filling the void spaces and; therefore, pore pressure buildup is undoubtedly minimized due to the high compressibility of air in relation to that of water. As the degree of saturation is increased

and some of the air in the pore spaces is replaced by water pore pressure buildup, caused by compression of the voids, becomes much greater.

This possible influence of pore pressure is seen in the second and third group of curves, shown in Figure 7, which represent tests on the mixtures at degrees of saturation of about 85 percent and 100 percent, respectively. The degree of saturation represents the ratio of the volume of water in the voids to the volume of the voids present in the misture. If the dry density is held constant for each material, the void ratios are the same, the volume of voids are the same and these voids contain the same amount of water. As a result, the fluid in the voids has the same initial value of compressibility. This compressibility of the fluid in the pore space will be given the designation C_w (9). Since the relative densities of the mixes with different grain size distributions are different, the ability for densification of each mixture varies; the ability for densification decreases as the relative density increases. Therefore, at low relative densities, densification is more active and pore pressure buildup is much greater as the pressure exerted on the void spaces is higher due to the increased tendency for densification. For this reason, the effective stress in the mixtures with low relative densities is lower than in the specimens with higher relative densities and failures of the specimens occur much more rapidly.

The above is best seen in the group of curves shown in Figure 7 in which the specimens were saturated to about 100 percent. At 1000 load cycles, the materials are progressively weaker in the order of A, C and B which corresponds to a decrease in relative density from 97.6 percent to 96.3 percent and finally 92.8 percent. The middle group of curves, which represent degrees of saturation of about 85 percent, exhibit more stability

due to their higher C_w value and corresponding lower values of pore pressure. The increase in the value of C_w is due to the increased proportion of air in the pore fluid mixture. It will be noted here that curve A-83 percent indicates an inconsistency in the theory presented above. Not only is it inconsistent in its position due to its assumed relative density but also because the average degree of saturation of 83 percent is much lower than that for the C gradation with 87 percent saturation. On a basis of degree of saturation alone, gradation B should be much stronger than gradation C. Reference to Figures A-1 and A-3 shows that the average dry densities for gradations A and C were 143.8 pcf and 144.3 pcf, respectively. As these dry densities would reverse the position of the magnitudes of relative density for the materials, this fact might explain the reason for the reversal in the trend of stability in the materials.

Another point of interest is the effect of densification on the compressibility of the fluid in the pore space, C_w. It is logical to assume that as densification progresses some of the air in the voids, being less viscous than the water, would be displaced by aggregate particles in preference to the water and would flow to less active portions of the specimen. The result is a decrease in void ratio and, if all the pore water remains, the degree of saturation of the active zone of the specimen would be increased. As the degree of saturation has a direct influence on the value of C_w, the magnitude of C_w would decrease thus reducing the effective stress and the specimen would become progressively weaker because of the reduction in effective stress. This decrease in C_w would also increase the rebound characteristics of the specimens as higher pore pressures would have to be relieved after the completion of each load cycle. This would account for an increase in rebound as the stress history progresses.

The increase in rebound during the loading cycles is illustrated for the gravel material in Figure 9. It will be noted that towards the end of each test made on the specimens saturated to about 85 percent and greater, the magnitude of rebound reaches a maximum value and then begins to decrease. This decrease is probably due to the extreme distortion of the specimens at this point in the test and the resulting large decreases in deviator stress due to the increase in cross-sectional area of the specimens. This can be substantiated by comparing the total specimen deflection at the approximate peak points of the rebound curves. From Figure 9, for those specimens saturated to about 85 percent, the maximum value of rebound occurs for gradations A, B and C at about 10,000, 5,000 and 40,000 load cycles, respectively. The total deformation found in Figure 7 for these gradations at 85 percent saturation is 0.42, 0.41 and 0.41 inches, respectively. As these values of total deflection are essentially equal, this would seem to substantiate the reasoning for the decrease in magnitude of rebound given above. Unfortunately, this same correlation is not present in the specimens saturated to about 100 percent. Further, no explanation has been found for the shape of the curves representing gradations A and B saturated to about 70 percent. A definite trend seems to be present even though the change in rebound for these specimens was less than 0.001 inch. One possibility could be that the specimens which exhibited this trend were approaching their fatigue limit, however, this cannot be substantiated.

The same reasoning given in the preceding paragraph would hold true for the initial values of C_w for the various saturation levels used. As the initial degree of saturation is increased in the specimen prior to the test, the initial value of C_w decreases thus increasing the resulting pore pressures in the specimen during test. Upon release of load, the increased

pore pressure causes greater rebound in the material initially saturated to the higher levels. This explains the increased magnitude of rebound as the initial saturation level used in the tests is increased. Three distinct changes in the magnitude of rebound are seen in Figure 9.

Figure 11 illustrates for the gravel the loss in strength which occurs as the degree of saturation is increased. The most alarming fact illustrated here is the tremendous decrease in stability which results when the degree of saturation is increased from about 85 percent to approximately 100 percent. It will be noted that the 6.2 percent and 11.5 percent gradations are almost identical as far as stability is concerned. Compare, for example, these two materials at 1000 load cycles with degrees of saturation approximating 70 percent. At about 95 percent saturation, the deflection of the 6.2 percent material falls in line with that expected had the 11.5 percent gradation been tested at this saturation level. The intermediate point on the 6.2 percent gradation curve also coincides with the expected deflection of the 11.5 percent gradation. This, of course, is to be expected as the relative densities of these two gradations are almost identical. The curve for the 9.1 percent gradation further illustrates the fact that a low relative density produces reduced stability. From the limited data available and discussed above, it would seem that, for the gradations tested, if the relative density of two materials are equal their stability will be about the same.

Figure 13 shows the influence of degree of saturation on the rebound characteristics of the gravel materials. The gradations containing 6.2 percent and 11.5 percent fines performed almost identically from the standpoint of rebound. As previously stated, the relative densities for these two gradations were almost the same. The only point which deviates from

this close relationship is the 6.2 percent gradation at the high degree of saturation. Only one test was performed at this saturation level so this point cannot be varified. The 9.1 percent gradation again produced the greatest rebound.

Performance of Crushed Stone

The preceding explanations given for the action of the gravel materials subjected to repeated stress histories are also applicable to the tests on the crushed stone materials. Due to the irregularity in degree of saturation and the inability to test these crushed stone specimens at high degrees of saturation, it was felt that an understanding of the actions of the two materials would be obtained if full attention were paid to the results obtained with the gravel material. However, a short discussion will be made relative to the results plotted in Figures 12 and 14 which summarize the behavior of the specimens of crushed stone that were tested.

An inconsistency appears in the data presented in Figure 12. It appears from the results shown that the 9.1 percent gradation at saturation levels above about 70 percent is a little more stable than the 6.2 percent gradation even though the latter has a relative density of 98.2 percent as compared to 90.0 percent for the 9.1 percent gradation. These two points on the 9.1 percent gradation curve are taken from results of single test specimens. Check specimens might have yielded slightly higher deflections which would have raised the position of this curve. As specimens with saturation levels above about 80 percent were rejected for reasons previously stated, the trends in stability at higher degrees of saturation were not determined; however, the slopes of the curves indicate that the stability would continue to decrease, as in the case of the gravel, as the saturation

level is increased. The curves in Figure 14 correlate well with the theory presented previously except for the one point on the 11.5 percent gradation curve at 70 percent saturation. The data for this point were taken from a single test which might account for the inconsistency. Again, details of rebound behavior at saturation levels above approximately 80 percent are not available. In a general way, rebound of the crushed stone specimens increases as the relative density of the specimen decreases. The trend might be attributed partly to the increase in fines, but this cannot be substantiated from the available test data. As the magnitude of rebound remained essentially constant for the 6.2 and 11.5 percent gradation of gravel which had about the same relative density, this effect on rebound due to an increase in fines in the crushed stone specimens is probably negligible. Therefore, the theory concerning the effect on rebound due to a change in relative density discussed in the section concerning the gravel specimens is also probably valid here.

Comparison of Gravel and Crushed Stone

It has been shown that relative density plays a dominate role in the performance of these materials. Therefore, a comparison of the performance of the gravel and crushed stone gradations should logically be based on equal values of relative density. Further, to eliminate the effects of variation in the percentage of fines, if these two materials could be compared at equal values of fines and also equal relative densities, the results should be the most realistic possible. Referring to Figure 15, it will be noted that the 6.2 percent gradation of gravel had a relative density of about 98 percent. This compares with the relative density used in the 6.2 percent gradation for the crushed stone. Also, the relative

density of the gravel containing 9.1 percent fines was about 93 percent as compared to the 9.1 percent gradation of crushed stone which had a relative density of 90 percent. These materials will be compared on this basis; that is, G-A will be compared with S-A and G-B will be compared with S-B.

Table 5 has been prepared from the information contained in Figures 7 and 8. It will be noted that two levels of saturation are used for comparison, approximately 70 percent and 85 percent. As the specimens tested at the higher saturation levels failed at relatively low magnitudes of load repetitions, all comparisons were made for the magnitude of deflection of the specimens at 10,000 load cycles. Here performance is measured as the ability of the triaxial specimen to withstand deformation under repeated stress application. It is interesting to note that at a high relative density (98 percent) and with a degree of saturation of about 70 percent, the crushed stone had deformed about 150 percent more than the gravel. At the higher saturation level (approximately 82 percent), the trend was reversed and the results show that the gravel had deformed about 25 percent more than the crushed stone. When the relative density was decreased to about 92 percent, the variation in stability of the two materials at the lower saturation level was much greater. At a degree of saturation of about 74 percent, the crushed stone deformed about 430 percent more than the gravel. As the degree of saturation was increased, the trend again reversed and the gravel deformed about 100 percent more than the crushed stone.

This table indicates several significant trends. At equal relative densities and at relatively low degrees of saturation, the gravel appears to have superior characteristics from the standpoint of stability. As the

TABLE 5

COMPARISON BETWEEN PERFORMANCE OF GRAVEL AND CRUSHED STONE SPECIMENS OF EQUIVALENT GRADATIONS

Selected Test Series	Average Degree of Saturation (%)	Average Relative Density (%)	Total Deflection at 10,000 Cycles (in.)
G-A	69	98	0.02
S-A	67	98	0.05
G-B	73	93	0.03
S-B	75	90	0.16
G-A	83	98	0.41
S-A	81	98	0.32
G-B	86	93	0.63
S-B	81	90	0.31

relative density is decreased, the superiority of the gravel increases significantly in magnitude. However, comparisons made for these two materials at the higher degrees of saturation (approximating 85 percent) indicates that this trend is no longer present. At a degree of saturation near 82 percent and a relative density of approximately 98 percent, the stability of the crushed stone material was slightly superior to that of the gravel and at a degree of saturation of about 84 percent, with a reduction in the level of relative density to about 92 percent, the crushed stone material's stability is about double that of the gravel material.

It should be pointed out that in the last comparison given above, the degree of saturation of the gravel at a relative density near 92 percent was five percent higher than that of the crushed stone. This explains in part the variation in stability of the two materials in this range of relative density. The data in Table 5 suggest that at low degrees of saturation, the gravel material is superior to the crushed stone at all levels of relative density. Furthermore, as the relative density of the two materials decreases, the difference in stability between the gravel and crushed stone increases significantly, the stability of the gravel being less affected by a change in relative density than that of the crushed stone. Further, at higher degrees of saturation, the variation in stability disappears and the materials begin to perform in the same manner.

The explanation for the above may lie in the differences found in the fine portion of each material. The fines in the crushed stone were composed solely of limestone screenings from the crusher whereas the fines used in the gravel material consisted of a low plasticity clay. Data from the AASHO Road Test show that the liquid limit of the fine fraction of the gravel was 19 percent and the plasticity index was three percent. The

crushed stone screenings were reported to exhibit no plasticity properties. At the lower level of saturation, the clay portion of the mixture contained little moisture and thus exhibited some cohesion. This caused a high bonding action throughout the specimen which resulted in high stability. However, as the degree of saturation of the specimen was increased, the fines had an increased moisture content. This increase in the level of moisture of the fines decreased their cohesion and thus reduced their effectiveness in bonding the larger particles together.

It is reasonable to expect that the angles of internal friction for the gravel and crushed stone compacted to the densities used in the AASHO Road Test would be in the same order of magnitude possibly varying by as much as ten degrees. As the gravel exhibits some cohesion, at low confining pressures its strength should be greater than that of the crushed stone material which is cohesionless. It is assumed that the stone, which is more angular than the gravel, possesses a higher friction angle than the gravel. Therefore, had the confining pressure of 15 psi, used throughout the testing program, been increased to a higher value, it seems likely that the differences in stability of the two materials would have been reduced and at high values of confining pressure, it is most likely that the crushed stone would have been the most stable of the two materials. Since confining pressures in excess of 15 psi probably did not exist in the base course of the pavement, the reported trends are considered to be realistic.

Laboratory Comparison of AASHO Gradations

Attention will now be directed to a discussion of the relative performance, found in the laboratory tests, of the materials having gradations the same as those actually used in the AASHO Road Test project the gravel in the test pavement contained 9.1 percent finer than a number 200 mesh sieve whereas the crushed stone contained 11.5 percent finer than a 200 mesh sieve.

For the range of confining pressure used, there appear to be three dominating factors which are involved in determining the relative stability of these two materials at these particular gradations. The first is degree of saturation, the second is the bonding effect of the fines, and the third factor is the relative density of the materials. The effect of these three factors on stability of the two materials are interrelated. As the degree of saturation in a specimen is increased, the moisture content of the clay fines in the gravel mixture increased, thus reducing the cohesion of the binder. As the fines contained in the crushed stone material did not exhibit significant cohesion, the stability of this material, from the standpoint of the binding qualities of the fines, does not change appreciably with moisture content. Considering relative density, the lower the relative density, the lower the value of stability. This, as has been previously discussed, is due to the increased ability of the material to densify which increases the pore pressures; thus reducing the stability of the specimen. Neglecting the change in the binding effect of the material as the degree of saturation is increased, the magnitude of difference in stability of both materials should remain constant. This is due to the fact that as the degree of saturation in both materials is increased but kept at equal values, the percentage of the volume of water to volume of voids will remain equal for each mixture. Therefore, the decrease in value of compressibility of the pore fluid, C_w, will remain constant for each mixture as the level of saturation is increased. Thus, the difference in stability should also remain in the same proportion as before.

A comparison of the relative densities of the two materials shows that the gravel had a relative density in the test road of 92.8 percent as compared to 83.8 percent for the crushed stone. From the standpoint of

relative density alone, this would indicate that regardless of the level of saturation, the gravel should exhibit the greatest stability. However, considering the effect of moisture on the fine portion of the gravel specimens, as the degree of saturation is increased, the stability of these specimens should decrease. When taken together, these properties should tend to produce a convergence in the levels of stability as the degree of saturation is increased. This can be seen in the data previously presented. From a comparison of the curves in Figures 11 and 12, it will be noted that the stability of the gravel is greater than that of the crushed stone at all levels of saturation plotted. However, when a comparison is made of the difference in deflection of the two materials with the deflection of the gravel material, it can be seen that this ratio decreases as the saturation level increases. This would indicate that the assumption presented above is correct. Further, at a degree of saturation of about 70 percent, the value of deflection of the gravel specimens was negligible as compared to that of the crushed stone specimens. It would therefore seem that if the two materials were found to have equal field saturation levels corresponding to about 70 percent, the gravel would perform in a superior manner. However, if the field degrees of saturation of these materials were both above 85 percent, both materials would probably be found to perform about the same.

In comparing the two curves for the AASHO gradations which are shown in Figures 13 and 14, it is interesting to note the difference in the values of rebound. At 75 percent saturation, the gravel rebound is approximately 0.009 inch and the crushed stone rebound is almost 0.011 inch, an increase in magnitude of over 20 percent. However, at a saturation level of 80 percent, the difference in magnitude of the two gradations is much less and

the trends indicate that the values of rebound might intersect in the vicinity of 85 percent saturation.

It would appear from these results that if these two materials, compacted to the density levels reported for the AASHO Road Test, retained equal degrees of saturation up to approximately 80 percent, the gravel would provide a better base course foundation not only from the standpoint of stability but also from that of resiliency which, in itself, may ultimately cause pavement failure through fatigue.

Comparison of Laboratory and Field Results

The following summary of comparisons between the field performance of the gravel and crushed stone base courses has been taken from information supplied by personnel associated with the AASHO Road Test. This information was supplied from observations and records available at the site. The discussion which follows concerns only the performance of the base courses underlying the asphalt surfaces. The thickness of each base course material was varied during construction so that the effects of thickness on performance of the asphaltic surface could be analyzed. The gravel base course varied in thickness from two to 18 inches and the crushed stone base course varied from two to 19 inches.

The pavement was opened to traffic in the fall of 1958 and during the first few months, very little distress was observed in the pavement sections overlying either base material. A little rutting occurred in the pavements overlying the gravel and crushed stone bases, but its magnitude was negligible. During early spring some map cracking became apparent in the pavements overlying the gravel bases and, it is interesting to note, the extensiveness of the map cracking did not seem to vary with the gravel base

thickness. By late spring many of the sections on gravel base, regardless of base thickness, had deteriorated. The map cracking became excessive and rutting occurred. At the completion of the scheduled test program, (winter of 1960), some of the thicker crushed stone base sections were still in service whereas many of the sections built over the gravel were out of test. In contrast to the performance found in the gravel base course sections, the stability of the crushed stone sections improved as the thickness of the base course increased. Also these sections, prior to failure, exhibited less fatigue cracking in the surface than did the gravel sections. Rutting seemed to be the controlling factor for failure in the crushed stone base sections.

It is of interest to compare the moisture contents and degrees of saturation of these two materials in the field. The moisture content and dry unit weight of test samples taken throughout the field testing program were supplied by the AASHO personnel. The degrees of saturation reported herein are computed values using the specific gravity found in laboratory tests on corresponding samples of the materials. The specific gravity values used for the gravel and crushed stone were 2.74 and 2.81, respectively.

The in-place field moisture contents at the time of construction were 6.0 percent for the gravel and 5.8 percent for the crushed stone. These moisture contents correspond to degrees of saturation of 94 percent for the gravel and 68 percent for the crushed stone. These moisture content determinations were made during the construction period sometime in the summer of 1958. Test trenches were dug during the spring of 1959 and further moisture contents were determined. It was found that the average moisture content of the gravel was now 5.5 percent and that of the crushed

stone 4.7 percent. These moisture contents correspond to degrees of saturation of 81 percent for the gravel and 56 percent for the crushed stone. Due to the failure of the gravel sections, further data on the moisture contents of the gravel base at later dates in the history of the test are not available. Further determinations of the moisture present in the crushed stone base were made in the spring and summer of 1960. In the spring, the average moisture content was 4.4 percent which corresponds to a degree of saturation of 55 percent. The tests made in the summer of 1960 indicated an average moisture content of 3.6 percent which corresponds to a degree of saturation of 44 percent.

To summarize, it is seen that the degree of saturation of the crushed stone at time of placement in the summer of 1958 was 68 percent. By the spring of 1959, the value had decreased to 56 percent and the following spring, its value was 55 percent. The determination made in the summer of 1960 indicated a degree of saturation of 44 percent. Corresponding to these figures, the gravel material was placed at a degree of saturation of 94 percent and when tested in the spring of 1959, the gravel was found to be 81 percent saturated.

It is interesting to note the difference in permeability of these two materials at the density levels at which they were placed in the field. From Figure 17, it will be seen that the coefficient of permeability of the gravel base was about 0.01 foot per day as compared to a value of about 8.5 feet per day for the crushed stone material.

The above data present a picture of the change in moisture conditions which took place in these materials over the life of the pavement sections. Referring to Figure 1, it will be noted that the gravel material contained an average of four percent finer than 0.02 millimeter whereas the crushed

stone material contained an average of five percent finer than 0.02 millimeter. Using the Corps of Engineers frost criteria, both of these materials fall into the F1 group and therefore are potentially frost-susceptible. However, as indicated by the high permeability of the crushed stone, the voids of the crushed stone mixture are no doubt too large to promote capillary rise. Furthermore, free water is quickly dissipated through drainage outlets provided at the shoulder base contact.

Since the gravel gradation falls into the F1 group and since its coefficient of permeability is very low, active frost action can be expected. Further, as this base is difficult to drain, free water resulting from surface infiltration will remain in the base for long periods of time.

The relative permeabilities of the two materials support the trends found in the variation of saturation and the performance of the pavements with time. Since the crushed stone was easily drained, it is evident that the initial saturation level of 68 percent decreased to a lower value during the ensuing months prior to winter. Since the material is not susceptible to frost action, the moisture in the crushed stone probably froze at this level of saturation and remained in this condition until spring. As the degree of saturation determined in the spring of 1959 was 56 percent and one percent lower the following spring, this probably corresponds to about the equilibrium moisture content for the compacted crushed stone. During the summer, it would be expected that evaporation would take place, thus reducing the moisture content as was indicated by the saturation level of 44 percent found in the summer of 1960.

Referring to the data concerning the gravel base, since the saturation level dropped from 94 percent at the time of construction to 81 percent the

following spring, some drainage or desiccation took place. Since the gravel gradation has a lower permeability and is frost susceptible, it would be logical to expect close to 100 percent saturation at the time of spring break-up. The period of time which elapsed between spring break-up and the date the moisture content determinations were made is not known and therefore it is impossible to estimate the approximate moisture level at the critical time of thaw. However, for purposes of comparison, a conservative estimate of the degree of saturation at the end of spring thaw would be at least 90 percent. This is contrasted to an estimated value of about 60 percent for the crushed stone at the time of thaw. Referring to Figures 11 and 12, it is interesting to compare the deflection of the test specimens at these saturation levels.

At a degree of saturation of 60 percent and a density of 141 pcf, the crushed stone deflected about 0.025 inch. Referring to the curve for the gravel, at a saturation level of 90 percent and a density of 145 pcf, the magnitude of deflection was about 0.30 inch. Further, referring to Figures 13 and 14, the rebound exhibited by the crushed stone was about 0.008 inch as compared to a rebound of 0.014 inch for the gravel. From these figures it would appear that, under the moisture conditions estimated for the two base course materials at the critical period of spring break-up, the crushed stone should perform better than the gravel. Also, the data indicate that the rebound of the gravel would be much greater than that of the crushed stone base.

These conclusions, concerning the performance of the laboratory specimens at the estimated levels of saturation that were found in the field, substantiate the method and progression of the pavement failures in the field. Using the gradations specified, compacted to the relative densities

previously mentioned and subjected to these repeated load stresses at the specified degrees of saturation, the data indicate that the crushed stone base should perform better than the gravel base.

It is difficult to draw definite conclusions regarding the superiority of either of these materials since their performance depends on many factors which are not inherent properties of the individual aggregate particles themselves. Such things as relative density, degree of saturation, frost susceptibility, permeability, type of binder, particle shape and gradation all contribute their share in determining the stability of a base course material.

SUMMARY

The following summary is based primarily upon the laboratory tests reported herein and is subject to the limitations brought out in the main body of the thesis. The statements are based, in part, upon the premise that the assigned rating of the crushed stone was higher than that of the gravel in the test pavement.

1. The gravel and crushed stone used as base materials in the AASHO Road Test special wedge sections contained about the optimum quantity of fines for ease of compaction. The optimum fines for the crushed stone was somewhat greater than for the gravel.
2. The compactive energy required to obtain a specified level of dry density was less in the case of the crushed stone than for the gravel.
3. A small change in the degree of saturation had a greater effect on the stability and rebound properties of the base materials than did a slight change in relative density.
4. The effect of a change in relative density on the magnitude of rebound was significant. As the compacted relative density was decreased, the rebound upon release of load increased.
5. At low degrees of saturation, the gravel material was found to be superior (insofar as stability-rebound characteristics are concerned) to the crushed stone at all levels of relative density. Further, as the relative density of the two materials was decreased,

the difference in stability between the gravel and crushed stone increased significantly. The stability of the gravel was less affected by a change in relative density than that of the crushed stone. At higher degrees of saturation, this variation in stability between the two materials seemed to disappear and they performed in a similar manner.

6. From the laboratory tests, it was found that the coefficient of permeability of the gravel at the gradation which was used in the field was significantly lower than that of the crushed stone. Grain size analysis indicated that the gravel was frost-susceptible. The data suggest further that the crushed stone was potentially susceptible to frost but that due to its relatively high coefficient of permeability it was not adversely affected by freezing temperatures. In addition, field moisture determinations showed that the gravel existed at a higher degree of saturation than the crushed stone base during the time traffic was on the road. This variation in degree of saturation in turn no doubt accounts for the major portion of the difference in field performance of the two materials.
7. It is not possible to draw generalized conclusions relative to the superiority of either of the materials as their performance depends on many factors which are not inherent properties of the mix itself. The data indicate that the primary reason for the improved performance in the pavement using crushed stone as a base as contrasted to the performance of the pavement having a gravel base was not due to the inherent strength characteristics of the base materials but was due rather to the external effects of

moisture on those strength properties.

The data further suggest that if the gravel studied existed in a road at a degree of saturation equal to that of the crushed stone, its performance would be at least equal to and, at low degrees of saturation, superior to that of the crushed stone.

SUGGESTIONS FOR FURTHER RESEARCH

1. Using the data presented in this thesis, it would be very helpful to apply visco-elastic theories to further clarify the reasoning for the performance of these two materials found in the laboratory tests.
2. As the behavior of pore pressure is considered to be of extreme importance in this type of study, it is recommended that additional tests be performed on the gravel and crushed stone materials for the purpose of varifying the suspected variation of pore pressures in the materials under repeated load conditions.
3. The research should be extended to include appropriate tests on the bituminous-stabilized and portland cement-stabilized base materials used in the AASHO Road Test for the purpose of providing an evaluation of their behavior under repeated loads. From this extended research program, an attempt should be made to correlate the performance of all four materials under a wider range of in-situ climatic conditions.
4. The need for fundamental research on the stress-strain characteristics of semi-flexible (composite) pavement structures is advisable.

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BIBLIOGRAPHY

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APPENDIX

Deflection and Rebound Histories

for

Gravel and Crushed Stone Specimens

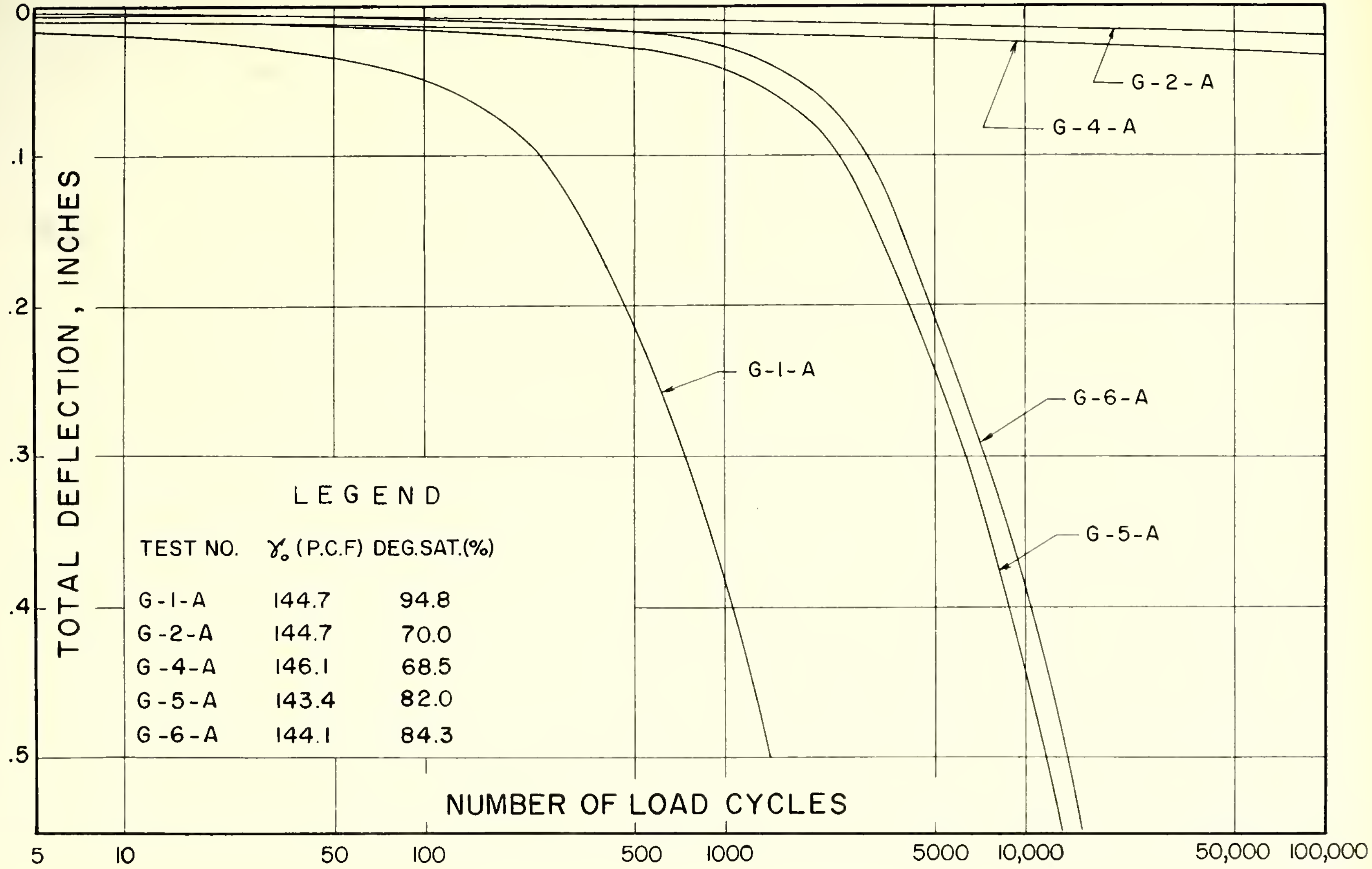


FIG. A-1 DEFLECTION HISTORY OF GRAVEL SPECIMENS — 6.2% PASSING NO.200 SIEVE

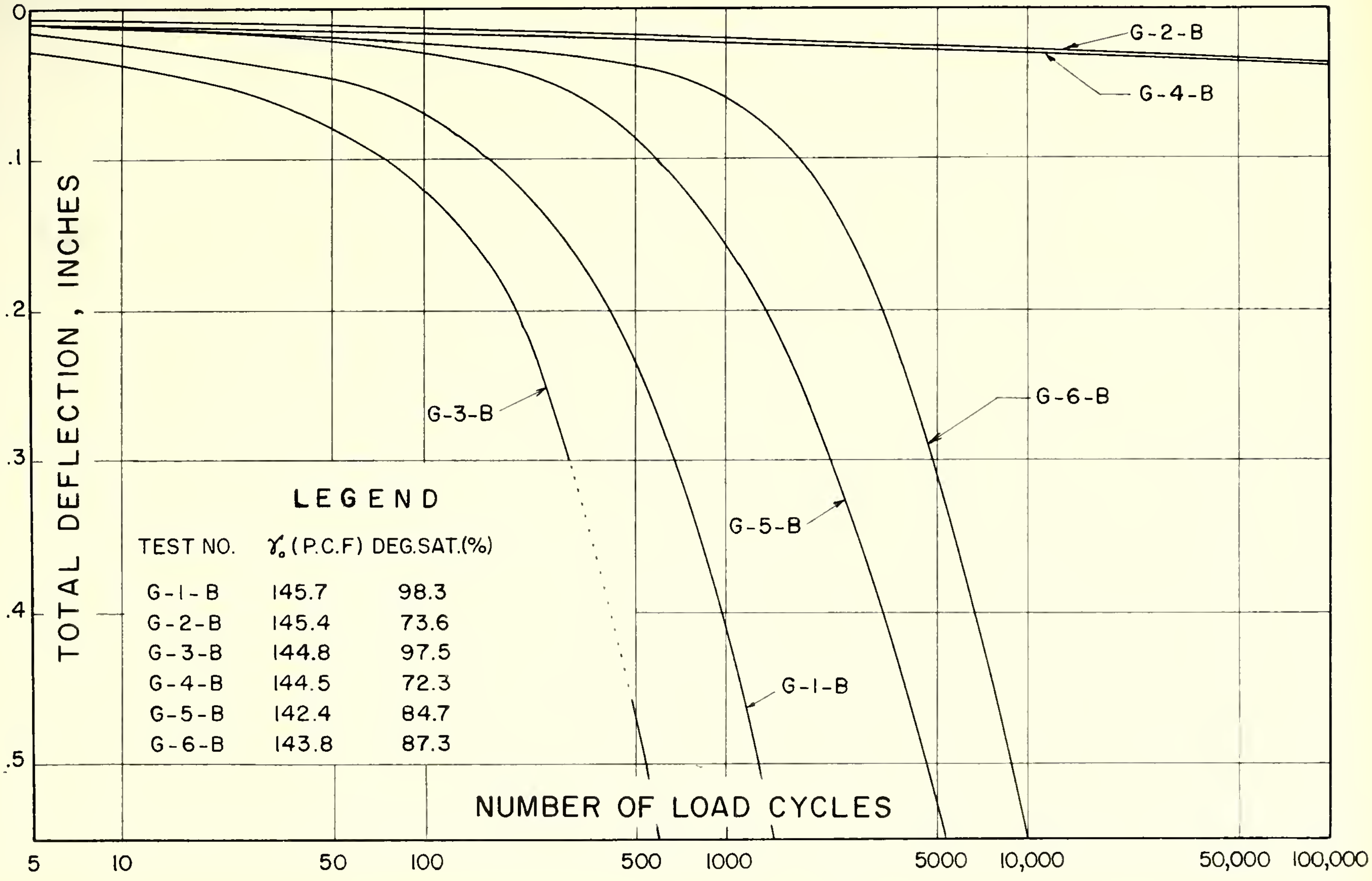


FIG. A-2 DEFLECTION HISTORY OF GRAVEL SPECIMENS — 9.1 % PASSING NO. 200 SIEVE

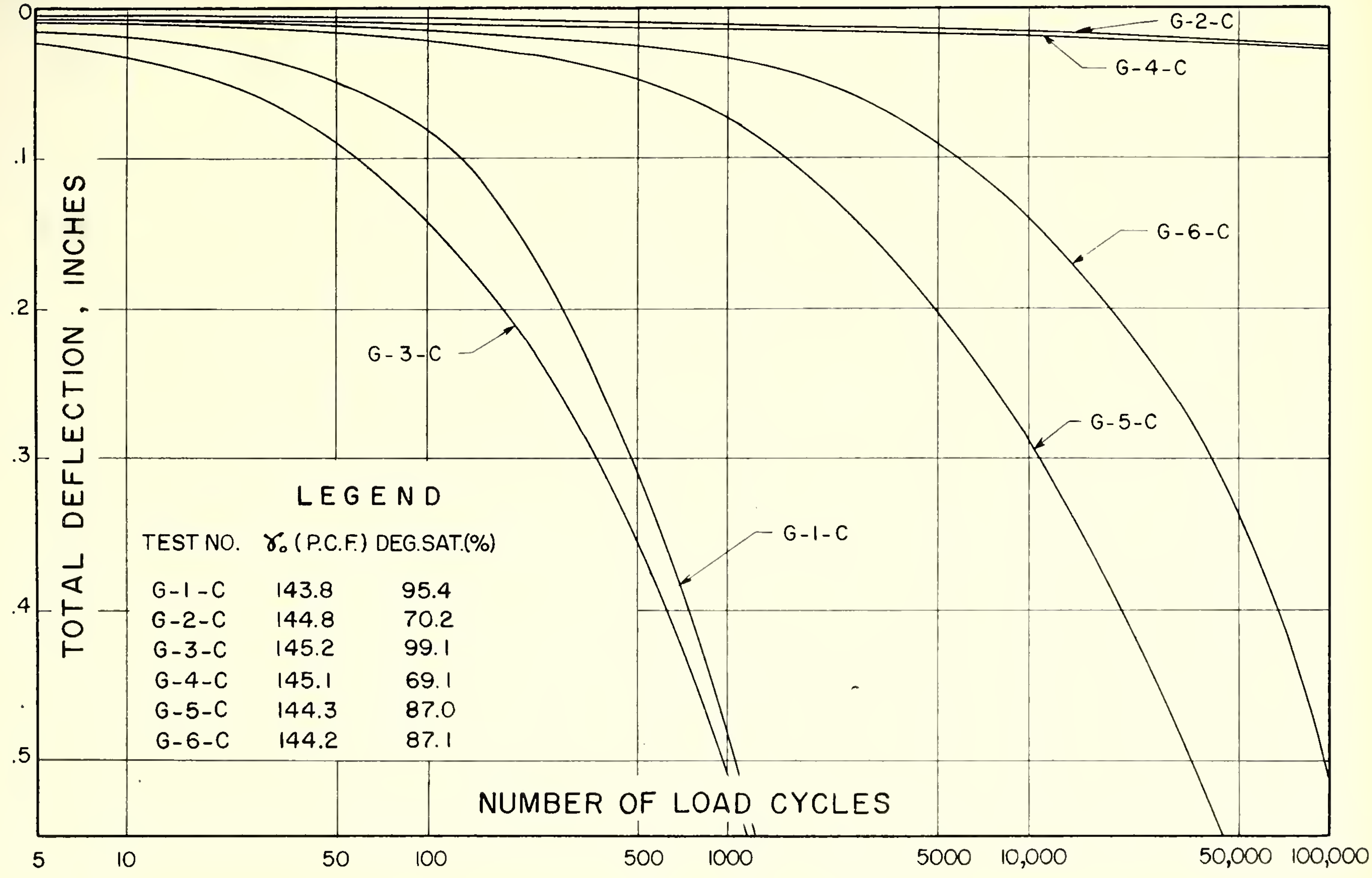


FIG. A-3 DEFLECTION HISTORY OF GRAVEL SPECIMENS — 11.5% PASSING NO. 200 SIEVE

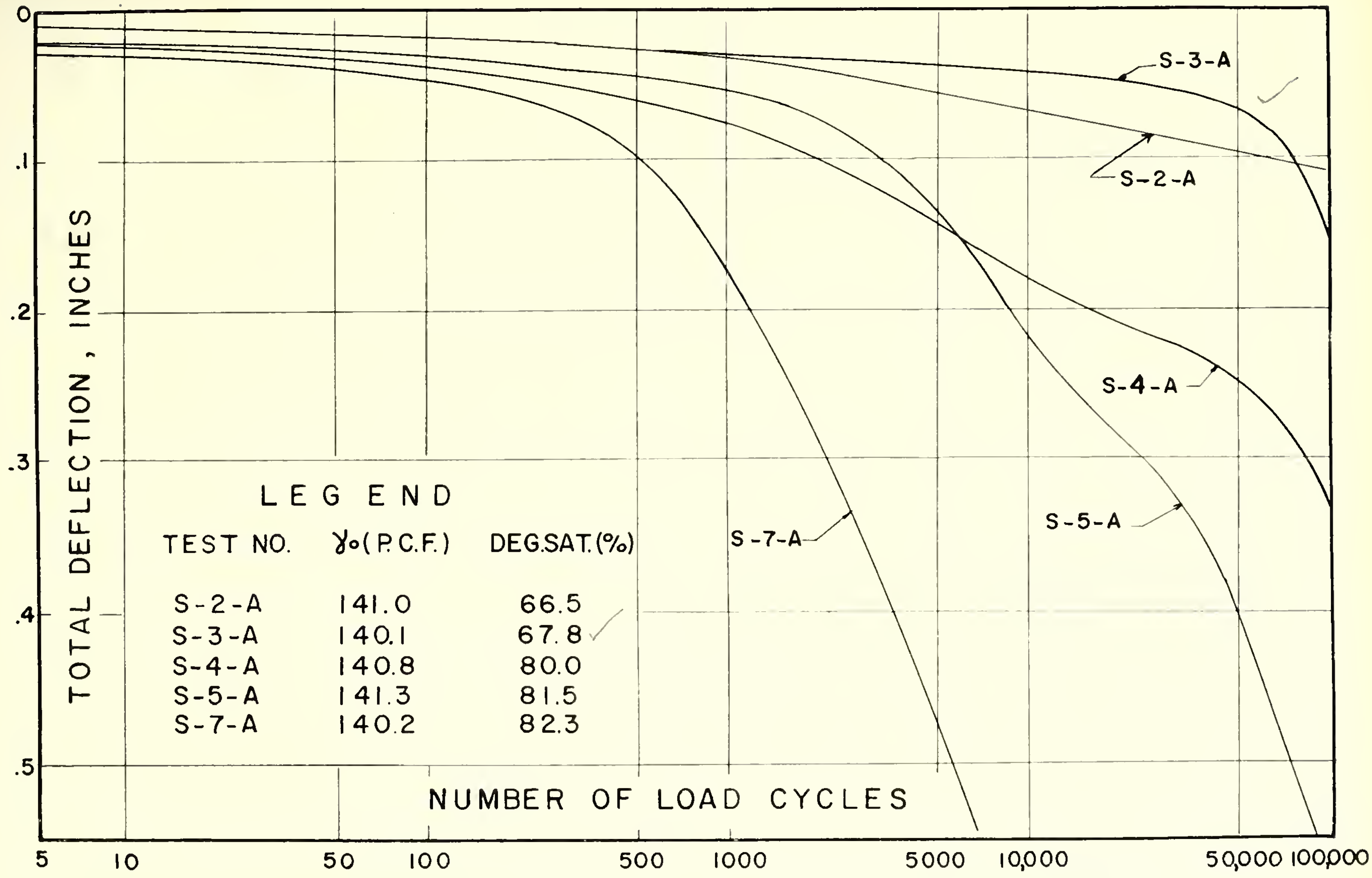
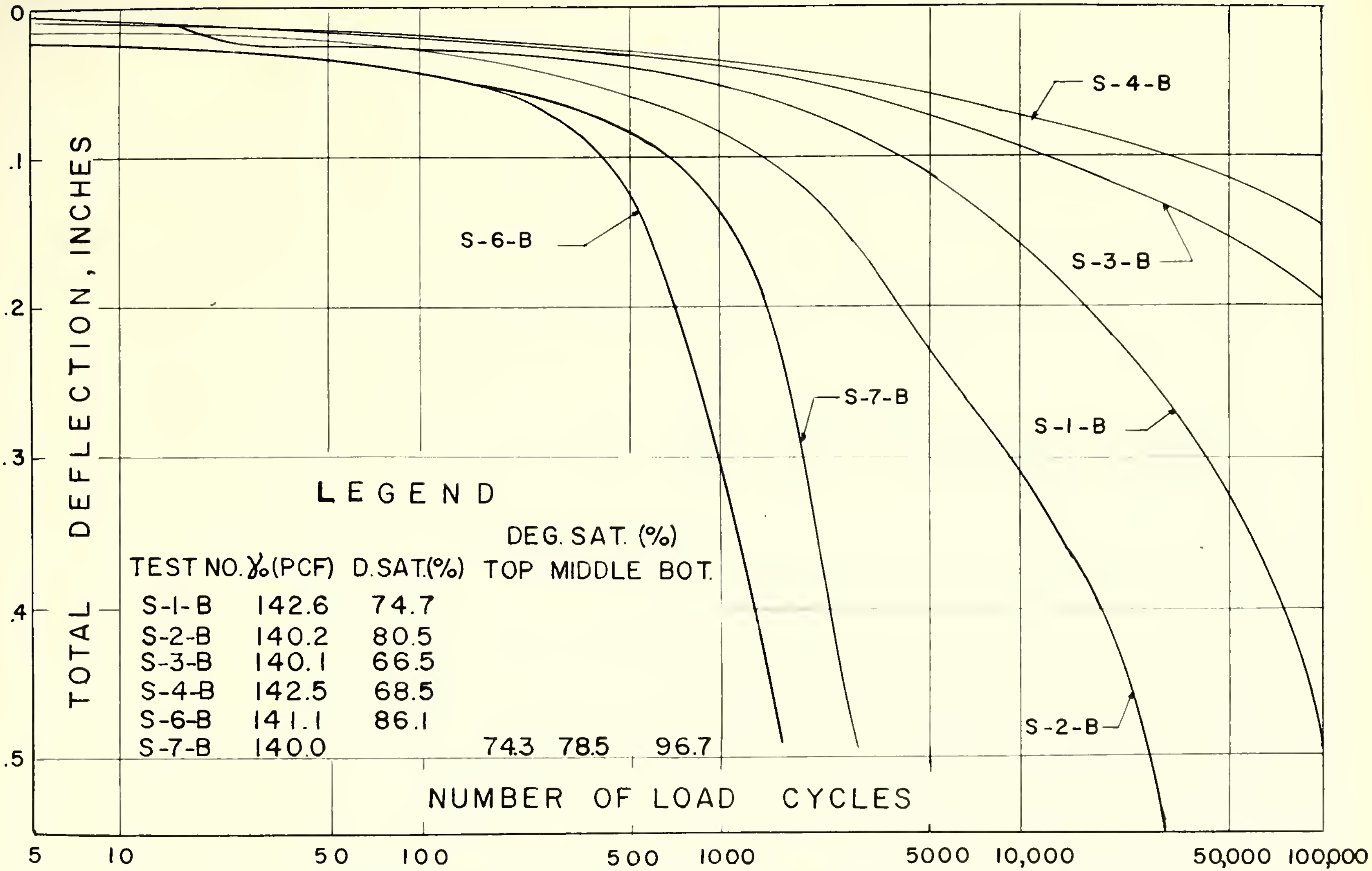


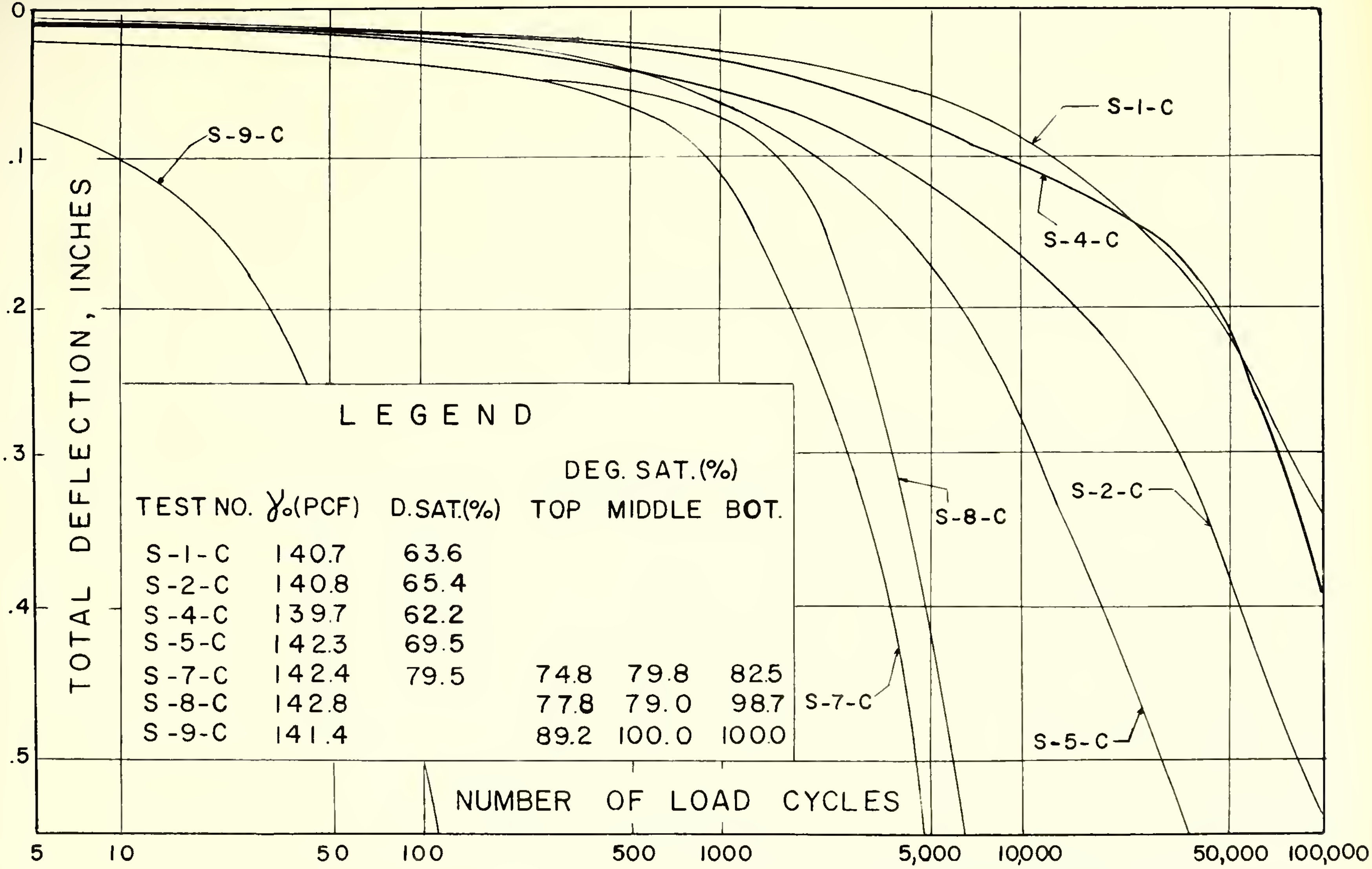
FIG. A-4 DEFLECTION HISTORY OF CRUSHED STONE SPECIMENS
6.1% PASSING NO.200 SIEVE



TEST NO.	γ_0(PCF)	D.SAT.(%)	DEG. SAT. (%) TOP	DEG. SAT. (%) MIDDLE	DEG. SAT. (%) BOT.
S-1-B	142.6	74.7			
S-2-B	140.2	80.5			
S-3-B	140.1	66.5			
S-4-B	142.5	68.5			
S-6-B	141.1	86.1			
S-7-B	140.0		74.3	78.5	96.7

FIG. A-5

DEFLECTION HISTORY OF CRUSHED STONE SPECIMENS
9.1% PASSING NO.200 SIEVE



LEGEND

			DEG. SAT. (%)		
TEST NO.	γ_o (PCF)	D.SAT.(%)	TOP	MIDDLE	BOT.
S-1-C	140.7	63.6			
S-2-C	140.8	65.4			
S-4-C	139.7	62.2			
S-5-C	142.3	69.5			
S-7-C	142.4	79.5	74.8	79.8	82.5
S-8-C	142.8		77.8	79.0	98.7
S-9-C	141.4		89.2	100.0	100.0

FIG. A-6 DEFLECTION HISTORY OF CRUSHED STONE SPECIMENS

11.5% PASSING NO. 200 SIEVE

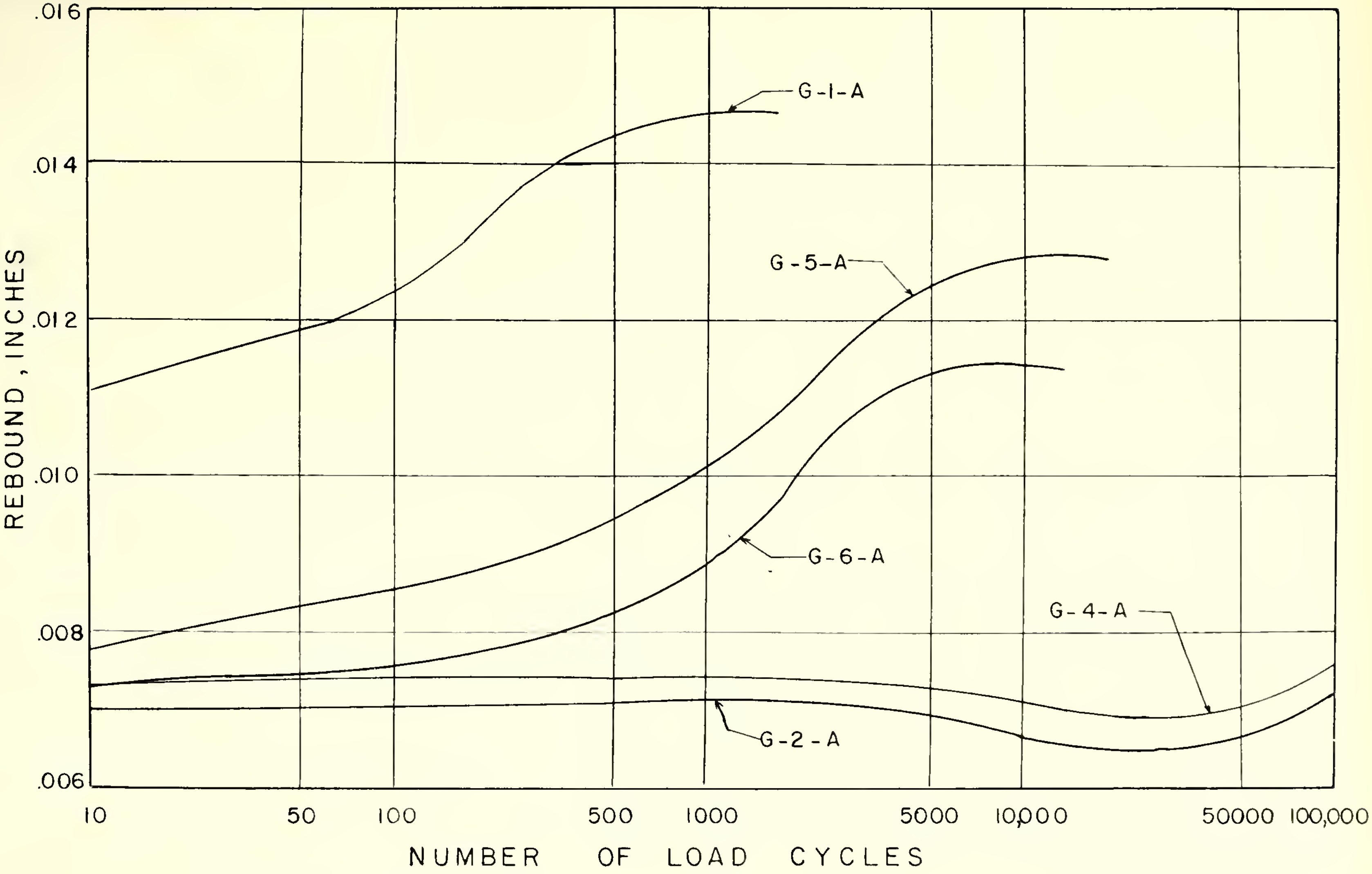


FIG. A-7 REBOUND HISTORY OF GRAVEL SPECIMENS — 6.1% PASSING NO. 200 SIEVE

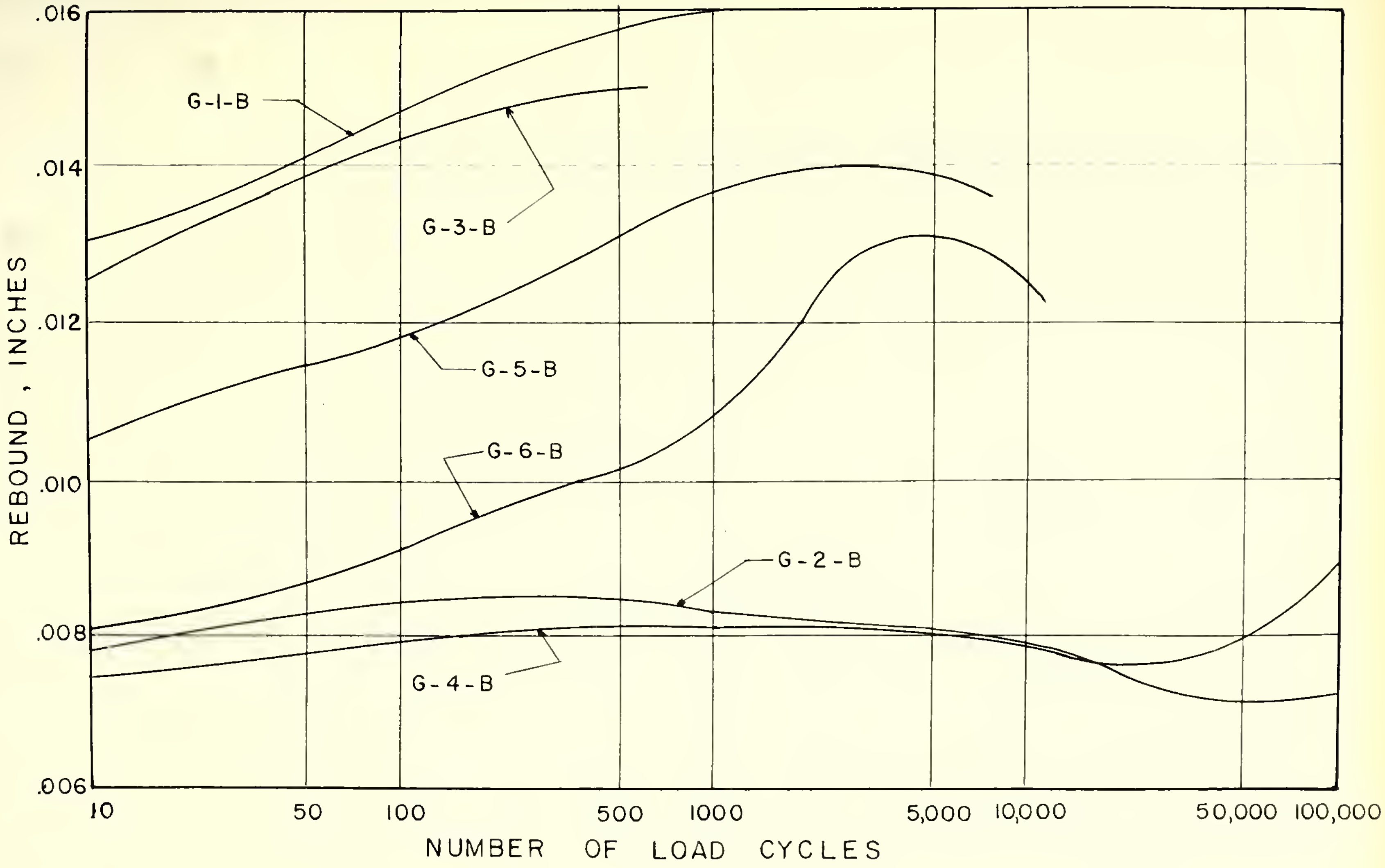


FIG. A-8 REBOUND HISTORY OF GRAVEL SPECIMENS — 9.1% PASSING NO. 200 SIEVE

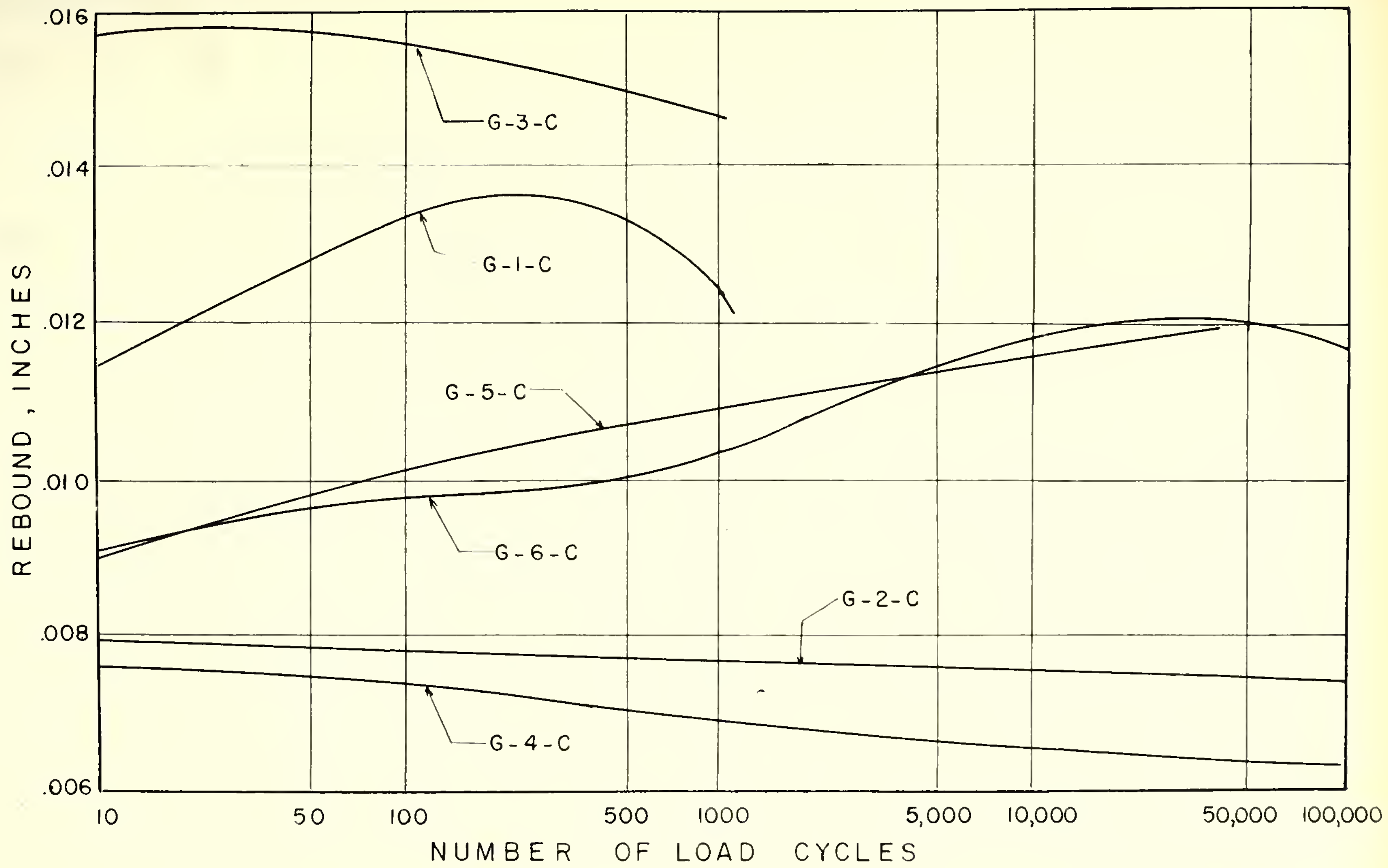


FIG. A-9 REBOUND HISTORY OF GRAVEL SPECIMENS — 11.5 % PASSING NO. 200 SIEVE

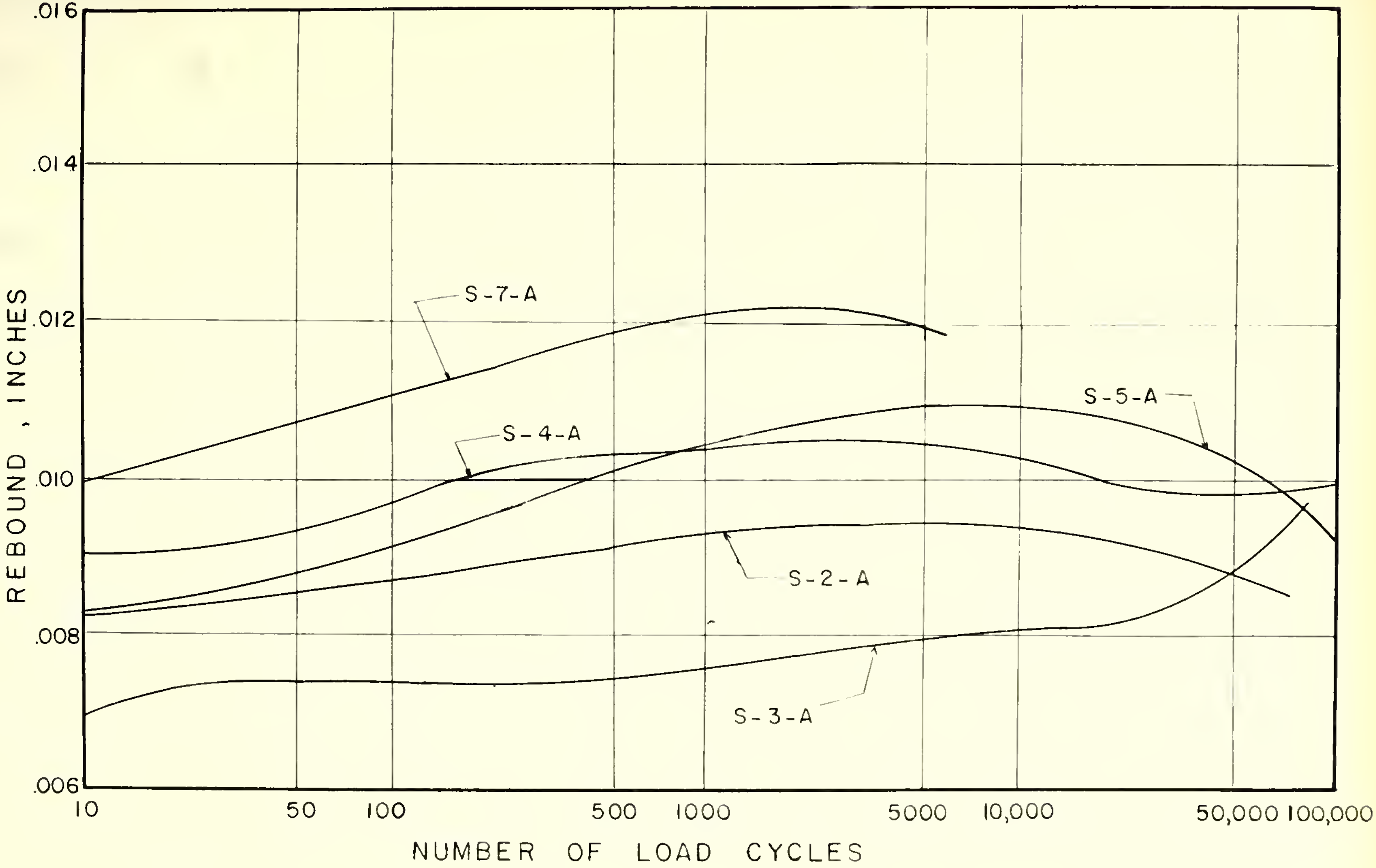


FIG. A-10

REBOUND HISTORY OF CRUSHED STONE SPECIMENS
6.1 % PASSING NO. 200 SIEVE

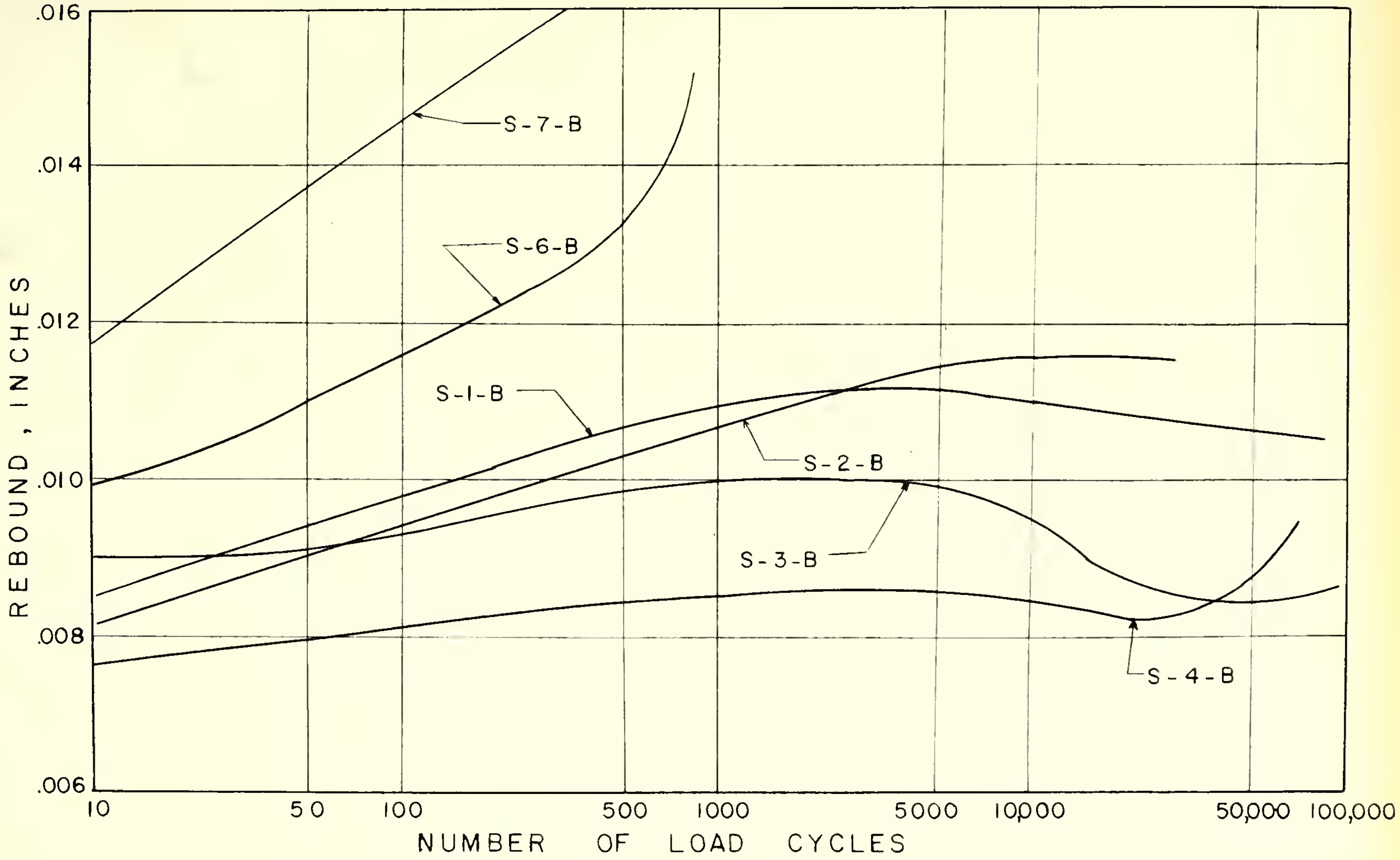


FIG. A-11 REBOUND HISTORY OF CRUSHED STONE SPECIMENS
9.1% PASSING NO.200 SIEVE

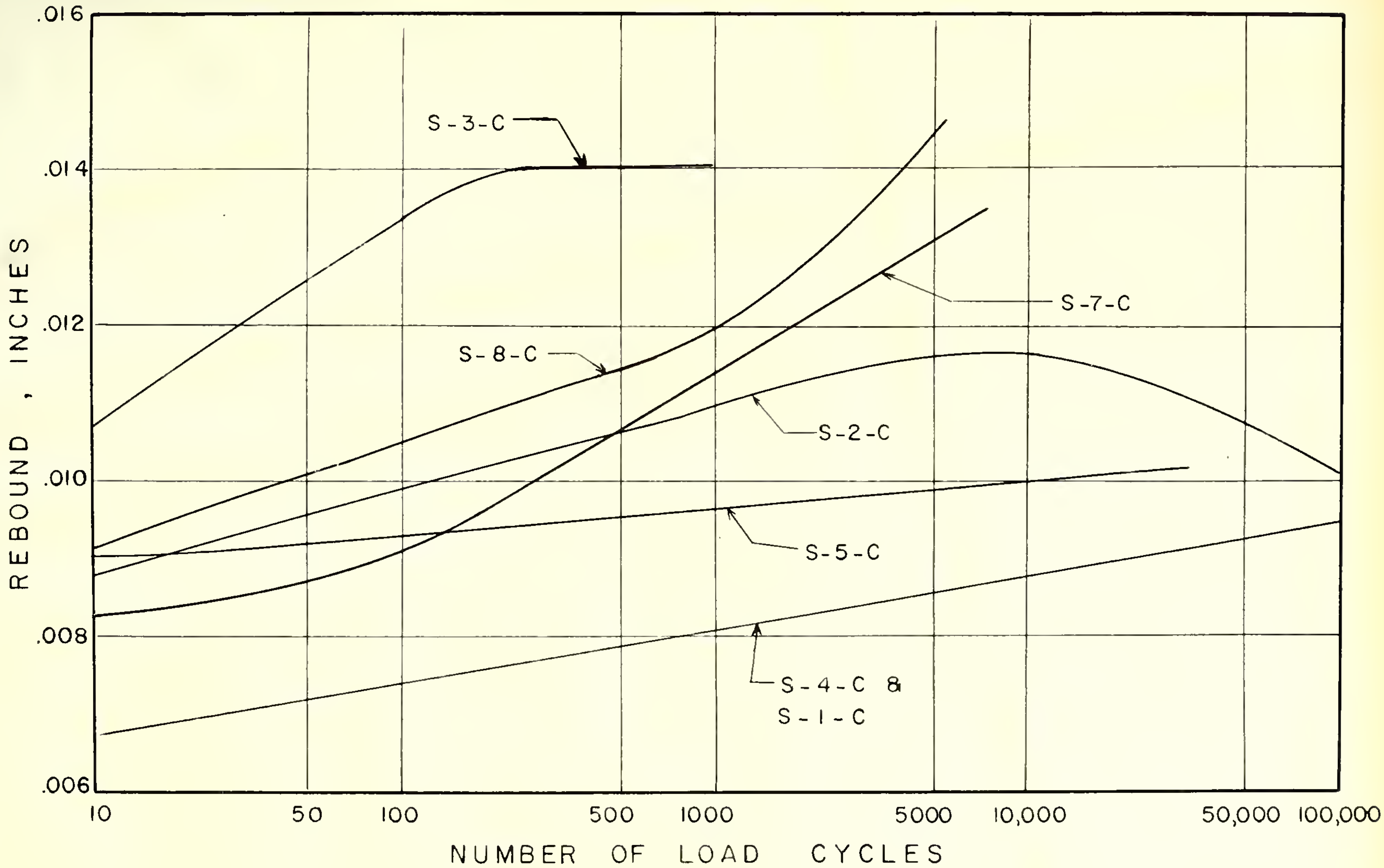


FIG. A-12

REBOUND HISTORY OF CRUSHED STONE SPECIMENS
11.5% PASSING NO.200 SIEVE